JOURNAL

OF

AMERICAN CONCRETE INSTITUTE

No. 3 JANUARY, 1914 VOL. I

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PUBLISHED AT THE OFFICE OF THE INSTITUTE
HARRISON BUILDING, PHILADELPHIA, PA.

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PURPOSE OF THE INSTITUTE.

It is the purpose of the Institute, by means of conventions and publications, to increase and disseminate knowledge in regard to the use of cement, concrete and allied products; to conduct research into their properties and uses; and to formulate recommended practice and standard specifications.

Its objects are purely educational.

JOURNAL
OF
AMERICAN CONCRETE INSTITUTE

FOUNDED 1905, AND INCORPORATED 1906, AS
NATIONAL ASSOCIATION OF CEMENT USERS
CHARTER AMENDED JULY 2, 1913

The Institute is not responsible, as a body, for the statements and opinions advanced in its publications

NO. 3 JANUARY, 1914 VOL. I

PROGRAM FOR TENTH ANNUAL CONVENTION.

MONDAY, FEBRUARY 16.

10.30 A. M.

Meeting of the Board of Direction, Auditorium Hotel.

2.00 P. M.

Meeting of Sectional Committees on:

Building Blocks and Cement Products.
Education.
Fireproofing.
Form of Specification.
Insurance.
Nomenclature.
Reinforced Concrete and Building Laws.
Reinforced Concrete Highway Bridges and Culverts.
Roads and Pavements.
Sidewalks and Floors.
Specifications and Methods of Tests for Concrete Materials.
Treatment of Concrete Surfaces.

8.00 P. M.

Formal Opening of the Convention, Auditorium Hotel.

Address of Welcome to the City of Chicago.

Wm. H. Sexton, Corporation Counsel, on behalf of the Honorable Carter Harrison, Mayer.

Address of Welcome on Behalf of the Architects.

Elmer C. Jensen, President, Illinois Chapter, American Institute of Architects, Chicago, Ill.

Address of Welcome on Behalf of the Engineering Interests.

W. H. Finley, Chief Engineer, Chicago and Northwestern Railway, Chicago, Ill.

Address of Welcome on Behalf of the Contracting Interests.

Response by the President.

Business Session.

TUESDAY, FEBRUARY 17.

9.00 A. M.

Sectional Meeting on Concrete Roads and Pavements and Highway Bridges and Culverts.

10.00 A. M.

Methods and Costs of Concrete Road Construction in Milwaukee County.

H. J. Kuelling, County Highway Commissioner, Milwaukee, Wis.

Concrete Road Construction in Maryland.

Henry G. Shirley, Chief Engineer, State Roads Commission, Baltimore, Md.

Expansion and Contraction of Concrete Roadways.

Rudolph J. Wig, Associate Engineer and Physicist, Bureau of Standards, Washington, D. C.

Some Requirements in Vibrolithic Concrete Pavement Construction.

R. C. Stubbs, Dallas, Texas.

Reinforced Concrete Roadways and Pavements.

B. S. Pease, Engineer, Reinforcement Department, American Steel and Wire Company, Chicago, Ill.

Report of Committee on Concrete Roads and Pavements Presenting Revised Specifications.

C. W. Boynton, Chairman.

Report of Committee on Reinforced Concrete Highway Bridges and Culverts.

Willis Whited, Chairman.

8.00 P. M.

Annual Address by the President.

Richard L. Humphrey, Consulting Engineer, Philadelphia, Pa.

Progress and Development of Concrete Work on the Delaware, Lackawanna and Western Railroad.

A. B. Cohen, Concrete Engineer, Hoboken, N. J.

Concrete, as Employed in the Construction of Wharves, Quay Walls and Dry Docks, at the Brooklyn Navy Yard.

F. R. Harris, Civil Engineer, U. S. Navy, Brooklyn, N. Y.

The Properties of Portland Cement Containing a High Percentage of Magnesia.

P. H. Bates, Chemist, Bureau of Standards, Pittsburgh, Pa.

WEDNESDAY, FEBRUARY 18.

9.00 A. M.

Sectional Meeting on Concrete Products, Blocks, Tile, Pipe, Fence Posts, etc.

10.00 A. M.

Joint Session with the Interstate Cement Tile Manufacturers Association.

Report of Committee on Building Blocks and Concrete Products.
C. K. Arp, Chairman.

Specifications for Drain Tile.
A. Marsten, Director, Iowa Engineering Experiment Station, Ames, Iowa.

The Present Status of the Investigation of the Effect of Alkali on Cement Pipe.
Rudolph J. Wig, Associate Engineer and Physicist, Bureau of Standards, Washington, D. C.

Result of Tests on Plain and Reinforced Concrete Tile.
George P. Dieckmann, Chief Chemist, Northwestern States Portland Cement Company, Mason City, Iowa.

Layout of Concrete Products Plants.
E. S. Hanson, Chicago, Ill.

8.00 P. M.

Modern Concrete Roof Without Forms.
J. E. Payne, Engineer, General Fireproofing Company, Youngstown, Ohio.

Combination of Reinforcement and Forms in Concrete Construction.
T. W. Murray, Manager, Trussed Concrete Steel Company, Detroit, Mich.

Steel Forms in Concrete Construction.
C. D. McArthur, Chief Engineer, Blaw Steel Construction Company, Pittsburgh, Pa.

The Problems of the Contractor.
L. C. Wason, President, Aberthaw Construction Company, Boston, Mass.

THURSDAY, FEBRUARY 19.

9.00 A. M.

Sectional Meeting on Reinforced Concrete and Nomenclature.

10.00 A. M.

Business Session.
Report of Board of Direction.
Election of Officers.
Revision of By-Laws.

Report of Committee on Reinforced Concrete and Building Laws.
Alfred E. Lindau, Chairman.

Report of Committee on Nomenclature.
F. C. Wight, Chairman.

Full Sized Tests and Their Value in Concrete Construction.
Burtis S. Brown, Consulting Engineer, Boston, Mass.

Test of a Reinforced Concrete Flat Slab Floor.
W. A. Slater, Engineering Experiment Station, University of Illinois, Urbana, Ill.

7.00 P. M.

Informal Dinner.

FRIDAY, FEBRUARY 20.

9.00 A. M.

Sectional Meeting on Concrete Materials and Surface Treatment.

10.00 A. M.

Report of Committee on Specifications and Methods of Tests for Concrete Materials.
Sanford E. Thompson, Chairman.

Some Comparative Corrosion Tests of Plastered Metal Lath—A Progress Report.
J. C. Pearson, Bureau of Standards, Washington, D. C.

Data on Lime Putty and Cream of Lime.
Cloyd M. Chapman, Engineer of Tests, Westinghouse, Church, Kerr & Company, New York, N. Y.

Report of Committee on Treatment of Concrete Surface, Presenting Revised Specifications for Stucco.
Leonard C. Wason, Chairman.

2.00 P. M.

Sanitary Surfaces for Concrete Floors.
T. Hugh Boorman, Consulting Engineer, New York, N. Y.

Post Molds and Manufacture of Reinforced Concrete Fence Posts.
G. H. Dougherty, D. and A. Post Mold Company, Three Rivers, Mich.

The Concrete Fence Post.
A. M. Smith, Ohio Post Mold Company, Toledo, O.

Discussion on Concrete Fence Posts.

CONVENTION NOTES.

Dinner.

The members should make it a point to attend the informal dinner on Thursday evening of the Convention. There will be a number of excellent speakers, special songs are being prepared and arrangements completed to make the occasion thoroughly

enjoyable. The dinner is under the charge of Mr. R. F. Hall, who has done so much in the past to make these annual functions of the Institute so successful.

Printed Copies of Matters Coming Up For Discussion at Convention.

1. The present By-Laws together with amendments appeared in No. 2 of the JOURNAL; reprints will not be available at the Convention.

2. Revision of the Standard Specifications of Portland Cement Stucco appeared in JOURNAL No. 2; only reprints of the present specifications will be available at the Convention.

3. Report of the Committee on Concrete Roadways and Pavements; separate reprints of this report will be available at the Convention.

4. Report of the Committee on Standard Methods of Tests and Specifications for Concrete Materials; report will be available in printed form at the Convention.

5. Report of Committee on Reinforced Concrete Highway Bridges and Culverts appears in this issue of the JOURNAL and will not be available as a separate reprint at the Convention.

NEW FEATURES IN THE JOURNAL.

In subsequent numbers of the JOURNAL will appear the following new features:

1. A review of books, pamphlets and other publications of an educational character relating to cement and concrete. For this purpose the Institute requests all interested to forward such books and pamphlets as issued in order that due notice may be given in JOURNAL under "Technical Publications."

2. Under "Trade Publications" brief references will be given to catalogs and other descriptive matter relating to machinery, materials and appliances used in concrete construction.

3. Arrangements are now being perfected by which there will shortly appear in the JOURNAL and each issue thereafter, a brief review of the patents issued during the current month.

It is the purpose to thus keep regularly before the members of the Institute the latest publications of all kinds in the field of technical and trade literature.

NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.

Concrete road construction is a very live topic at the present time and information as to such roads is eagerly sought by all those who have to do with permanent road construction. The National Conference on Concrete Road Construction, which convenes in Chicago at the Auditorium Hotel, February 12, 13 and 14, is highly important and from present indications will doubtless be productive of much good. The activities of the Committees and the wide interest shown in this gathering assure its success and there is no doubt that its sessions will be extremely interesting and profitable. A complete program is enclosed herewith.

The Conference opens on February 12 at 2 P. M. at the Auditorium Hotel and subsequent sessions will be held at 10 A. M. and 2 P. M. Addresses will be made on

Financing Permanent Roads.
Can a Rural Community Afford Permanent Roads?
The Concrete Road System of Wayne County, Mich.
Development of Concrete Roads in United States.
Experiments with Concrete for Roads Conducted by the United States Office of Public Roads.
Concrete Road Construction in Ohio.

Committees will report on

Contraction and Expansion of Concrete Roads.
Joints for Concrete Roads.
Aggregates for Concrete Roads.
Preparation and Treatment of Sub-grade for Concrete Roads.
Reinforcement of Concrete Roads.
Methods and Cost of Repairing and Maintaining.
Shoulders for Concrete Roads.
Bituminous Surfaces for Concrete Roads.
Finishing and Curing Concrete Road Surfaces.
Economic Methods of Handling and Hauling Materials.
Mixing and Placing Materials for Concrete Roads.
Cost of Constructing Concrete Roads.
Thickness, Crown and Grades for Concrete Roads.
Proportion and Consistency of Materials for Concrete Roads.
Form of Specifications for Concrete Roads.

INSTITUTE NOTES.

Board of Direction.

In carrying out the spirit of the proposed changes in the By-Laws the Board of Direction at a recent meeting appointed a Committee on Nomination of Officers which, according to the By-Laws, will be announced at the opening session of the Convention. This Committee is actively investigating the merits of the various candidates for the offices of President, one Vice-President, Treasurer and one Director from each of Districts 2, 3 and 6.

A Finance Committee was appointed from the members of the Board to analyze and digest the finances of the Institute for the purpose of preparing a budget of expenditures for the current year, which will be embodied in the report of the Board to be presented at the Annual Convention.

For the purpose of determining ways and means of giving adequate publicity to the work of the Institute, the Board decided to create a Committee on Publicity. This Committee is in process of formation under the chairmanship of Mr. E. J. Mehren.

Binding Volume VIII of Proceedings.

Volume VIII covering the Proceedings of the Kansas City Convention, is in the hands of the printers. The volume will be sent to the members in paper covers unless cloth or half morocco binding is ordered before March 1, accompanied by remittance to cover the extra charge of 50 cents or $1.00, respectively.

Reduced Price For Back Volumes.

Members of the Institute can obtain copies of back publications at the following special prices:

Vol. I, 50 cents; Vols. II to VI, $1.50 each; all in paper binding. For cloth or half morocco binding there is an additional charge per volume of 50 cents or $1.00. Vol. VII is available only in cloth binding, $2.00, or half morocco, $2.50

REPORT OF COMMITTEE ON REINFORCED CONCRETE HIGHWAY BRIDGES AND CULVERTS.

It has seemed best to the Committee to cover certain points in specifications for Reinforced Concrete Highway Bridges and Culverts by definite specifications, but certain other points must be varied according to the special circumstances of the case and, therefore, what is considered to be the best engineering practice is stated and recommended for adoption by the public. If any of the other national associations, of recognized standing, take up the same work, it would seem well to co-operate with them. It has been the policy of the Committee to adopt the standard specifications or recommended practice of any national society whose authority was generally admitted.

The following specifications and recommendations are submitted to the Institute, not as final, nor even as voicing the unanimous opinion of the members of the Committee, but merely as preliminary suggestions, in the hope of bringing out the discussion of doubtful points and information regarding questions which the Committee could not answer. Any information or criticism will be thankfully received and carefully considered.

Specifications.

1. The specifications for *Reinforcement* and *Cement* should conform to those of the Institute.

2. The specifications for *Aggregates* should conform to those of this Institute.

3. The *Unit Stresses* in reinforced concrete, and reinforcement, should conform to the recommended practice of this Institute.

Recommended Practice.

DEAD LOADS.

4. The following are given as average weights per cu. ft. of the materials mentioned, but if the weights of the materials to be actually used are definitely known to be different from those given, the correct weights should be used.

Materials	Lbs. per Cu. Ft.
Earth filling	110
Plain concrete	150
Reinforced concrete	155
Steel	490
Cast iron	450
Vitrified brick	140
Common brick	125
Granite and limestone masonry	165
Sandstone	160
Macadam-Telford	150
Pine, fir, etc	42
Oak and yellow pine	48
Creosoted	60

5. The weight imposed by *earth filling* should, in ordinary cases, be considered as including all filling included between vertical planes passing through the faces of the abutments.

If, however, the height of the fill exceeds about two-thirds the distance face to face of the abutments, the live load may be neglected and a very considerable proportion of the weight of the filling considered will be supported by friction between it and the approach filling. The amount of load thus transmitted is greatly affected by the cohesion of the soil, of which there is nearly always more or less.*

This whole subject requires further investigation.

LIVE LOADS.

Class A Bridges—Main thoroughfares leading from large towns.

6. In view of the extensive introduction of the *heavy motor trucks and traction engines*, and the probable general use of such vehicles in the future, it is recommended that bridges on main thoroughfares and other roads which are likely to be used for heavy hauling, be designed to carry 20-ton trucks, with axles about 10 ft. c. to c., 14 tons on rear axle and 6 tons on fore axle; wheels about 5 ft. c. to c. Outside of the large cities it is recommended that only one such vehicle be assumed to be on the bridge at any one time, the liklihood of more than one being on the bridge, in a position to produce maximum stresses at the same time, is so

*This paragraph is not approved by Mr. Lemuel Holmes.

remote that this assumption is considered safe. It is advised that such very heavy oads be considered as occupying only the ordinary width of the road, about 8 ft. in width and about 35 ft. in length. Congested traffic of heavily loaded wagons or motor trucks will rarely impose a load of more than 100 lb. per sq. ft., over a considerable area. The above mentioned 20-ton truck gives a load of about 140 lb. per sq. ft., on the area actually occupied, but it is considered extravagant to assume that a large bridge is covered with such heavy loads. 100 lb. per sq. ft. is thought ample to assume for the loading of spans more than 60 ft. long, in designing the trusses or main girders. It is thought to be safe to reduce this assumed load in the case of longer spans, to the following amounts:

Length of Span, ft.	Assumed Load, lb. per sq. ft.
80	90
100	80
125	75
200 and over	70

With all intermediate spans in proportion.

7. *Sidewalk Loads.*—The greatest load that is liable to be imposed on a bridge sidewalk, occurs when there is some excitement in the neighborhood, which attracts a large crowd, and for which the bridge affords an especially good point of view. In that case the crowd forms a compact mass, against the railing, not more than 4 ft. deep, making a load seldom exceeding 100 lb. per sq. ft., over a very considerable space. The remaining portion of the sidewalk may be covered by a moving crowd which can scarcely weigh more than 40 lb. per sq. ft. It may be advisable, sometimes, to so design sidewalk slabs, that if a street car or motor truck accidently gets upon the sidewalk, it will not go through. Such accidents are so rare, that it is thought safe to allow materials to be stressed somewhat beyond the elastic limit in such cases.

8. *Class "B" Bridges.*—Although it is impossible to determine beforehand, especially in the newer parts of the country, whether any given road is to be used for heavy traffic, it seems extravagant, at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 15-ton trucks, with axles 10 ft. apart, 5 tons on the front and

10 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended, that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 90 lb. per sq. ft., for spans up to 60 ft.

9. The longer spans, the trusses and main girders should be designed for the following loads:

Length of Span ft.	Assumed Load lb. per sq. ft.
80	80
100	70
125	65
150	60
200 and over	55

With intermediate spans in proportion.

10. *Sidewalks.*—Sidewalks should be designed to carry the same loads as in the case of Class "A" Bridges.

Special Bridges.

11. City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., require special consideration and should, of course, be designed to carry any load which can reasonably be expected to pass over them, bearing in mind the likelihood of heavy traction engines and motor trucks coming into extensive use in the not distant future.

Stringer Loading.

12. The maximum stress in a stringer, due to a wheel load, occurs evidently when the wheel is directly over it. It is not thought proper to assume any distribution of the load to adjacent stringers, unless the bottom reinforcement in the slab is made continuous. In that case the distribution is proportional to the relative stiffness of the slab and the stringers, said stiffnesses being proportional to the moments of inertia, multiplied by the modulus of elasticity of materials and inversely proportional to the cube of the span. In determining this distribution, due account must be taken of the fact that deflection of the slab

decreases toward the end of the stringers, and also of the fact, that whatever load is carried to the adjacent stringers, deflects them also. It is therefore recommended that wherever practicable the bottom reinforcement of slabs be made continuous over the stringers.

Slab Loading.

13. The distribution in a direction parallel to the supports of a concentrated load resting on a slab, supported at two opposite edges only, evidently depends upon the same principles as those mentioned under *Stringer Loading.* The main difference being that what corresponds to the stringer in the former case is of indefinite width in the present case. Adequate theoretical investigations of this question appear to be lacking. Some excellent experimental work has been done but not of sufficient extent to really prove much. Your Committee hopes to make a more complete report on this subject later. For the present it seems fair to assume that the distribution each side of a concentrated load is equal to about one-third the length of the span, and that the cross reinforcement should be designed accordingly, which would require it to have an area of at least one-half of the principal reinforcement. The distribution of a concentrated load through earth filling on the top of a slab, does not appear to be very well understood, the Committee has not, thus far, been able to find any records of adequate theoretical or experimental investigation of the subject. It is hoped to obtain more data later.

Bridges Carrying Electric Cars.

14. Electric traction is still in its infancy and nobody is able to forecast its future development. It seems probable, however, that it will not be profitable to run cars weighing more than 50 tons each, at a speed that would be permitted on any public road. If very high speeds are desired, the traction company will doubtless be required to operate over its own right-of-way. It is recommended that bridges carrying either urban or inter-urban electric cars be designed to carry 50-ton cars on two trucks, spaced 30 ft. c. to c., each truck having two axles spaced 7 ft. c. to c. The Committee sees no reason for changing the customary practice of assuming that an axle load is distributed over 3 ties.

Loading on Arches.

15. The deflection of an arch being much less than that of a beam of the same length, the method recommended for determining the lateral distribution of a concentrated load over arch sheeting appears to be different from the distribution over flat slabs. It seems doubtful if the distribution in each direction can be greater than twice the thickness of the arch sheeting. This question should be investigated.

Stresses in Arches.

16. As all arches that are not provided with hinges, act as elastic arches until cracks are formed, due to excessive tension at some point or points in the concrete, it is manifestly proper to calculate the stresses in them according to the elastic theory.

17. As concrete is a very poor conductor of heat, it is not thought necessary in calculating reinforced concrete arches, to assume so much variation in temperature as is usual in designing steel structures, although the outside layers of concrete are of nearly the same temperature as the surrounding air, and those layers are stressed more heavily than any of the others, it is thought that an extreme variation of about 80 deg. in the Northern States is sufficient to allow for, in any case, and that can be reduced if the arch ring is thicker or if there is much earth filling in the spandrels.

BEARING POWER OF PILES.

18. The usual formula for the safe bearing power of wooden piles is:

$$B=\frac{2WH}{S+1}$$

in which, W = Weight of hammer in pounds,
H = Height of fall of hammer in feet,
S = Penetration in inches per blow, average of last few blows.

19. If reinforced concrete piles are molded before being driven, the head of the pile is usually cushioned more or less, and the pile is generally much heavier in proportion to the weight of the hammer than is the case with wooden piles. It is recommended that the hammer should be at least as heavy as the pile.

20. If concrete piles are molded in place, measures should be taken to prevent damage to them by the driving of neighboring piles or otherwise.

21. The Committee hopes to make some further investigations in the subject before submitting a final report.

BEARING POWER OF SOILS.

22. This subject is under an investigation by a committee from the American Society of Civil Engineers. It may be some years before their final report is submitted and even that will be subject to revision from time to time, as human knowledge is extended. In the meantime the Committee would submit the following preliminary table:—

Material	Safe Bearing Power, Tons Per Sq. Ft.
Quicksands and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1 to 3
Moderately dry clay confined	2 to 4
Dry stiff clay	4 to 6
Sand confined	2 to 6
Sand compact and cemented	4 to 8
Gravel cemented	8 to 12
Rock	25 to 200

Foundations should be carried down below frost unless they are on rock and thoroughly drained. Soil that contains the roots of plants is generally compressible. Undisturbed soil is much less compressible than filling or similar soil, even though it has been in place many years. The bearing power of sand, gravel and dry clay increases rapidly with the depth below the surface of the ground. None but the smallest structures should be founded on earth filling.

24. The pressure of earth against abutments, wingwalls and retaining walls varies so widely with the character and condition of the earth in question, that nothing more than a few general suggestions can be given. It is hoped that the Committee from the Society of Civil Engineers, which is studying the bearing power of soils, will take up this subject also. In ordinary cases where the filling is well drained its pressure will seldom be more than that of a liquid weighing 25 lb. per cu. ft., and is frequently much less.

The pressure due to land slides is often several times this amount. If the filling is clayey and is allowed to become waterlogged, its lateral pressure is greatly increased. The effect of the freezing of the filling must also be considered.

REQUIRED WATERWAY.

25. The usual formula for waterway for culverts is:

$a = CA^{\frac{3}{4}}$

in which,

a = Required area of culvert in sq. ft.

A = Drainage area in acres,

C = a constant, depending on the length and character of the drainage area and may vary from about .3 to 2.0 in regions where the mean annual rainfall is about 50 in.

The capacity of the culvert will be much greater if the wing-walls are flush with the abutments and flare about 30 deg., the sides and bottom of the culvert are smooth and straight, and the bottom has a good slope.

If reliable information covering a number of years can be obtained regarding the adequacy of the old bridge crossing the same stream, it is much more useful in determining the size of a culvert or bridge than any formula,

Respectfully submitted by the Committee,

WILLIS WHITED, *Chairman.*
LEMUEL HOLMES.
A. N. JOHNSON.
A. M. LOVIS.
HENRY H. QUIMBY.

SYNOPSIS OF RECENT ARTICLES ON CONCRETE.

Reviewed January 1 to 31, 1914.

The synopses given below are for the purpose of keeping the members of the Institute in touch with the leading articles of interest appearing in the many technical journals and is not a complete review of the journal. The publications regularly reviewed appear below and the number given corresponds to the number following the title of the paper. The length of the article in pages, whether illustrated or not, is noted, and the publications reviewed in which it appears:

1. Canadian Engineer, Toronto, Ontario, Canada.
2. Cement, New York, N. Y.
3. Cement Era, Chicago, Ill.
4. Cement World, Chicago, Ill.
5. Concrete Age, Atlanta, Ga.
6. Concrete Era, Los Angeles, Cal.
7. Concrete-Cement Age, Detroit, Mich.
8. Concrete and Constructional Engineering, London, England.
9. Engineering and Contracting, Chicago, Ill.
10. Engineering News, New York, N. Y.
11. Engineering Record, New York, N. Y.
12. Good Roads Magazine, New York, N. Y.
13. Portland Cement, Kansas City, Mo.
14. Rock Products, Chicago, Ill.
15. Proceedings American Railway Engineering Association, Chicago, Ill.
16. Western Contractor, Record Building, Kansas City, Mo.
17. Journal National Fire Protection Association, Boston, Mass.
18. Journal Western Society of Engineers, Chicago, Ill.
19. Proceedings American Society of Civil Engineers, New York, N. Y.
20. Proceedings American Society of Municipal Improvements, New York.
21. Proceedings American Society for Testing Materials, Philadelphia, Pa.
22. Proceedings Engineers' Club of Philadelphia, Philadelphia, Pa.
23. Proceedings Engineers' Society of Western Pennsylvania, Pittsburgh, Pa.
24. Professional Memoirs, Corps of Engrs., U. S. A., Washington, D. C.
25. Proceedings Institution of Civil Engineers, London, England.
26. Transactions, Canadian Society of Civil Engineers, Montreal, Quebec, Canada.
27. Railway Age Gazette, Chicago, Ill.
28. Better Roads, Jamestown, Ohio.

MATERIALS

Effect of Hydrated Lime on Portland Cement Mortars, by H. S. Spackman. (11) *Jan. 3, 2 pp. ill.* Changes in physical characteristics determined by tests with curves of results. The object was to determine how much lime can be added to cement mortars to increase the plasticity.

Some Fallacies in Cement Testing. (1) *Jan. 29, 3 pp.* Discussion of the various tests for cement.

DESIGN AND TESTS

Analysis of Reinforced Concrete Chimneys under Earthquake Stresses, by J. A. Griffin. (11) *Jan. 10, 1 1/2 pp.* Discussion of maximum stresses, points of breakage and formula for calculating principal dimensions of chimneys.

Designing Water Conduits of Large Diameter, by A. C. Janni. (10) *Jan. 8, 3 pp. ill.* Discussion of stresses in reinforced concrete pipes with formulas.

A Big Column Failure, by C. I. Auten. (10) *Jan. 29, 1 p. ill.* Description and cause of failure of concrete surrounding a structural steel column.

The Relation of Strength to the Temperature in Green Concrete. (7) *Jan., 2 pp.* Discussion and suggestion for tests together with table of strengths.

Steel Stresses in Flat Slabs, by Dr. H. T. Eddy. (19) *Jan., 50 pp. ill.* Discussion of stresses and results of tests on actual building.

HOMES

Small Fireproof Garages. (4) *Jan., 3 pp. ill.* (7) *Jan., 1 p. ill.* Design and construction.

Decorative Possibilities of Concrete. (5) *Jan., 3 pp. ill.* Suggestions for artistic use of concrete.

Tufa Concrete and Steel Forms in House Construction. (7) *Jan., 2 1/2 pp. ill.* Details of construction of cottage.

Concrete in Small Domestic Buildings. (8) *Jan., 6 pp.* Extracts from report of English Government Committee on use and cost of concrete for cheap cottages.

BUILDINGS

A New Type of Reinforced Concrete Floor Construction. (10) *Jan. 1, 2 pp. ill.* Description of ribbed-slab type of floor construction in Railway Exchange Building, St. Louis, 21 stories high and 228 x 271 ft. in plan.

Steel Forms in Building Construction. (4) *Jan., 7 pp. ill.* Description of standard types of adjustable steel forms and methods of using them.

Concrete Building for United States Bureau of Engraving and Printing at Washington. (3) *Jan., 2 pp. ill.* Construction of building 505 ft. long, 300 ft. deep and 5 stories high.

Concrete Capitol for Utah. (3) *Jan., 3 pp. ill.* Construction of building 155 x 400 ft., 4 stories with dome 164 ft. high of reinforced concrete.

Reinforced Concrete in the British Museum, London. (8) *Jan., 9 pp. ill.* Construction of building 320 ft. long, 50 ft. wide and 80 ft. high.

Reinforced Concrete in the New Law Courts at Kingston, Jamaica. (8) *Jan., 8 pp. ill.* Design and construction of building 258 ft. long, 136 ft. wide and 45 ft. high.

Reinforced Concrete Roof Trusses and Cantilevered Balconies. (11) *Jan. 31, 2/3 pp. ill.* Design and construction of cantilevered platforms in 6-story building 232 x 164 ft. of the Ford Motor Company, Chicago.

BRIDGES AND RAILROAD WORK

Boylston Street Subway in Boston. (11) *Jan. 3, 3 pp. ill.* General description and construction of a reinforced two-track subway 8000 ft. long.

Girard Point Grain Elevator. (11) *Jan. 10, 2 1/2 pp. ill.* Layout and construction of one million bushel grain elevator at Girard Point, Pa., consisting of 24 concrete tanks 13 ft. in diameter and 72 1/2 ft. high, together with 54, 15 ft. diameter tanks 96 ft. high.

Ortonville-Milbank Alignment Revision. (11) *Jan. 17, 1 p. ill.* Reinforced concrete work in improvement of Chicago, Milwaukee and St. Paul Railway, on about 11 miles of road.

Alternate Designs for New James River Bridge, Richmond, Va. (10) *¼ pp. ill.* Description of 3 designs submitted in competition for bridge 3479 ft. long.

Design and Construction of the St. Croix River Bridge at Hudson, Wis. (9) *Jan. 7, 2 pp. ill.* Description of reinforced concrete cylindrical piers 5 ft. diameter, 61 ft. high and reinforced concrete retaining walls.

Concrete Tile Floor Construction of Reinforced Concrete Highway Viaduct. (9) *Jan. 21, 1 p. ill.*

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Northern Pacific Ice House at Pasco, Wash. (27) *Jan. 23, 3 pp. ill.* Construction of reinforced concrete ice house 483 ft. long, 94 ft. wide and 41 ft. high.

Concrete Steel Bridge in Texas, by W. C. Davidson. (4) *Jan., 2 pp. ill.* Design and construction of bridge 30 ft. plan and 24 ft. wide.

A Concrete Arched Bridge at the San Diego Exposition. (7) *Jan., 3 pp. ill.* Description and construction of bridge consisting of seven 56-ft. spans and two approach girders on each end.

Skylights in Jersey City Train Shed. (11) *Jan. 31, 2/3 pp. ill.* Design of roof and skylights for Terminal Station of Central Railroad of New Jersey.

RESERVOIRS, DAMS, CANALS, ETC.

Constructing the Kensico Dam, by Geo. T. Seabury. (11) *Jan. 10, 4 pp. ill.* Design and construction of cyclopian masonry dam, 310 ft. high and 1843 ft. long.

Reinforced Concrete Standpipe with Sliding Base, by Wm. Mueser. (11) *Jan. 10, 2 pp. ill.* Design and construction of cylindrical standpipe 100 ft. high and 40 ft. diameter of reinforced concrete.

Big Creek Hydroelectric Development. (11) *Jan. 10, 3 1/2 pp. ill.* Design and construction of 4 dams having an aggregate length of 2700 ft. and ranging in height up to 135 ft.

Jordan River Dam on Van Couver Island. (11) *Jan. 17, 3 pp. ill.* Design and construction of hollow reinforced concrete dam 126 ft. high and 756 ft. long.

Coon Rapids Hydroelectric Development. (11) *Jan. 17, 4 pp. ill.* Design and construction of reinforced concrete dam 1175 ft. long.

Collapse of Groined Arched Roof at the Belmont Filters, Philadelphia. (10) *Jan. 15, 4 pp. ill.* (11) *Jan. 24, 1 1/2 pp. ill.* Conditions and probable cause of failure of roof 125 ft. long after being in service for 9 years.

Failure of Stony River Dam. (10) *Jan. 22, 3 pp. ill* (11) *Jan. 24, 4 pp. ill.* Description and cause of failure of hollow reinforced concrete dam 1065 ft. long and 51 ft. high.

Construction Features of the Stony River Hollow Concrete Dam, by G. H. Bayles. (10) *Jan. 22, 7 pp. ill.* Design and construction of dam.

Tallulah River Developmen : of the Georgia Railway and Power Co. (10) *Jan. 29, 7 pp. ill.* Construction of hollow reinforced concrete dam 90 ft. high and 700 ft. long and another 135 ft. high and 444 ft. long. The latter of the gravity type. Highway bridge 12 ft. wide passes over dam. Construction of tunnel lined with concrete 6670 ft. long.

Design of Reinforced Concrete Settlin : Reservoir. (1) *Jan. 15, 3 pp. ill.* (9) *Jan. 21, 3 pp. ill.* Details of design of reservoir 156 ft. diameter, 20 ft. high.

Power Development at Cedar Rapids, Quebec. (1) *3 pp. ill.* Construction of concrete power house, 663 ft. long, 140 ft. wide and 50 ft. high.

Concrete on Baltimore Water Project. (3) *Jan., 4 pp. ill.* Description of concrete work covering dam, half mile conduit and concrete-lined water tunnel.

Grouting Cut-off for the Estacata Dam. (19) *Jan., 35 pp. ill.* Description of grouting foundations for heavy dam.

Concrete Troughs on the Barge Canal, by Geo. C. Mills. (11) *Jan. 31, 2 pp. ill.* Construction of 3 high embankments together with description of a break and temporary repair.

The Panama Canal. (9) *Jan. 7, 47 pp. ill.* General description of Canal, construction plants and costs for various parts of the work.

ROADS AND PAVEMENTS

Byberry and Bensalem Service Test Road, Philadelphia, Pa. (9) *Jan. 14, 7 1/2 pp.* Description of work and detail costs.

Discussion of Best Practice in Concrete Road Construction. (7) *Jan., 4 pp.*

Concrete Road in Iowa. (3) *2 pp. ill.* Construction and detail costs of road 16 ft. wide, one mile long.

SEWERS, PIPE, TILE, ETC.

Hamilton West End Sewage Disposal Works, by E. E. Ellis. (1) *Jan. 22, 6 pp. ill.* Design and construction of plant.

Tests Made on Pipe for Pressure Lines. (3) *Jan., 1 p.* Results of tests made on reinforced concrete pipe 48 to 84 in. diameter.

Strength of Large Pipes, by D. R. Cooper. (11) *Jan. 31, 4 pp. ill.* Discussion of stresses in circular conduits.

CONCRETE PRODUCTS

Concrete Trolley Poles in Cleveland. (5) *Jan., 1/2 p.* Description of 750 concrete poles together with cost.

MISCELLANEOUS

Concrete and Reinforced Concrete in 1913, by Sanford E. Thompson. (11) *Jan. 3, 1 p.* Review of important work done during year 1913.

Problems of the Contractor. (9) *Jan. 14 and 28, 7 1/2 pp. ill.* (11) *Jan. 3, 1 1/2 pp.* Abstract of paper by L. C. Wason on the various phases of contracting work.

New Garbage and Refuse Incinerator for San Francisco, by L. H. Nishkian and H. A. Minton. (10) *Jan. 15, 6 1/2 pp. ill.* Description of plant. Design and construction of reinforced concrete chimney 150 ft. high, 9 ft. diameter and reinforced concrete garbage pit.

Reinforced Concrete Panorama, Zoological Gardens. (8) *Jan., 11 pp. ill.* Description and construction of structures for displaying animals in their natural surroundings, such as artificial goat hills, etc.

A Reinforced Concrete Chimney. (8) *Jan., 4 pp. ill.* Design and construction of chimney 220 ft. high with hollow air space.

Design of Mine Shaft Linings, by W. A. Weldin. (2) *Dec., 48 pp. ill.* Use of concrete. Bibliography on concrete lining appended.

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PART II. Papers, Discussions and Reports

TESTS TO DETERMINE LATERAL DISTRIBUTION OF STRESSES IN WIDE REINFORCED CONCRETE BEAMS.

By W. A. Slater.*

1. TESTS.

Object of Tests.—It is apparent that in a slab supported on two edges and having a load concentrated on an element g extending in the direction of the span as shown in Fig. 9, the deflection of this element will be greater than that of an element c parallel to it but at some distance from g. The proportion of the total load carried by the various elements will be a function of their relative deflections and also of the curvature of the strip g relative to that of a strip at right angles to it on the center line of the span. An analysis showing the action of such a beam must be based on assumptions which are of doubtful value even if the conditions of loading and support are accurately known. In practice there will be little uniformity in these respects. The slab forming the cover of a box culvert will be supported on rigid abutment walls and the slab of a beam girder and slab floor system will be supported on beams which deflect at the center causing a larger reaction at the end of the strip c and a smaller one at the end of the strip g than would occur if the supports were rigid. The relative deflections and reactions will also be affected by the properties of the sections involved, by the moduli of elasticity in shear and in tension and by the length, width and depth of the beam considered. Tests of reinforced concrete wide beams indicate that the load distribution is different also at different stages of the test even after failure of the concrete in tension. In view of the complexity of the problem and of the inter-dependence of the variables involved a thoroughly rational analysis to determine the distribution of stresses seems out of the question, though analytical considerations must form a guide for the carrying out of an experimental investigation. Such an investigation has been

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in progress at the Engineering Experiment Station of the University of Illinois for the past three years. Some of the results of this investigation and the methods of making the investigation form the subject matter of this paper.

Limitations of the Investigation.—The limitations of such an investigation are numerous. To obtain purely experimental results having any considerable range of applicability it is necessary on account of the large number of variables involved to make a great many more tests than has been practicable so far. It is necessary then in generalizing from the results to use judgment and some speculation as to what would be the effect of varying conditions other than those varied in the tests. For example; the tests so far made have been mostly upon beams of one span and one depth, and of varying widths. To apply the results to beams having other spans and other depths it is necessary to make assumptions as to the manner in which these variables enter.

A fact which limits the progress in tests on reinforced concrete test pieces is the impracticability of profiting by the information gained from one test in designing the next one. All specimens must be tested at the same age, usually 60 days, hence, in order to profit fully by one test in designing the next it must be 60 days between tests and this would be impracticably slow. Therefore, it is necessary to design a whole series of tests at a time. For instance in a series of tests made early in this investigation it was found that many of the beams were failing by diagonal tension and hence were not showing what their strength was against transverse stresses. To guard against diagonal tension failures in the next series of tests, bars were bent up and a weakness in bond strength was introduced which, in the light of tests made in the previous year, was unexpected. Hence although the tests made are believed to be of value, these weaknesses reduce the number of tests from which valuable conclusions may be drawn.

Description and Scope of Tests.—In this paper the two terms longitudinal and transverse are frequently used and refer to directions parallel with and at right angles respectively to the span. The term longitudinal reinforcement is used to mean the reinforcement in the direction of the span even though this be the short dimension of the beam.

Three series of tests on wide reinforced concrete beams have been made at the University of Illinois Engineering Experiment Station. These series were completed in 1910, 1911 and 1912 respectively, and the results formed subject matter for the bachelor theses of Messrs. K. E. Robinson, E. J. Schell and L. L. Livingston, who graduated in the civil engineering course at the University of Illinois in 1910, 1911 and 1912 respectively.

In making the tests the loads were applied through small rectangular blocks of cast iron or steel embedded in plaster of Paris in order to distribute the load uniformly over the area of the loading block. The centers of the loading blocks in all of the tests lay upon the longitudinal center line of the beam, and in all but two of the tests at the one-third points of the span. An effort was made to provide rigid supports though it is believed in some of the tests this condition was not obtained. Fig. 1 shows the range in width of loads and in width of beams.

In the series of tests of 1910 measurements were taken of deflections at various points of the beam. Fig. 2 shows a beam in the testing machine and the instruments in place for measuring deflections. The frame shown in Fig. 2 is attached to the beam at points on the sides immediately over the supports and at about the height of the neutral surface. Except for the rising of the corners from the supports the deflections are measured from a plane approximately parallel with the position of the neutral surface before testing. Most of these beams had a rigid support but were cushioned on a rubber hose as shown in Fig. 2. In all the tests of the 1911 series and of the 1912 series, measurements were made of deformations in the reinforcement and in some cases of that in the concrete. The measurements were taken in such positions as to show the distribution of the reinforcement stresses laterally from the center. The measurements of deformations in reinforcement and in concrete were made by means of a Berry extensometer (Fig. 3). This instrument and its use have been previously described by the writer.*

The beams of the 1911 series were tested in an inverted position in order to facilitate the taking of deformation measurements. Fig. 4 shows one of these beams in the testing machine. In these

*See Proceedings, Vol. VIII. The Testing of Reinforced Concrete Buildings

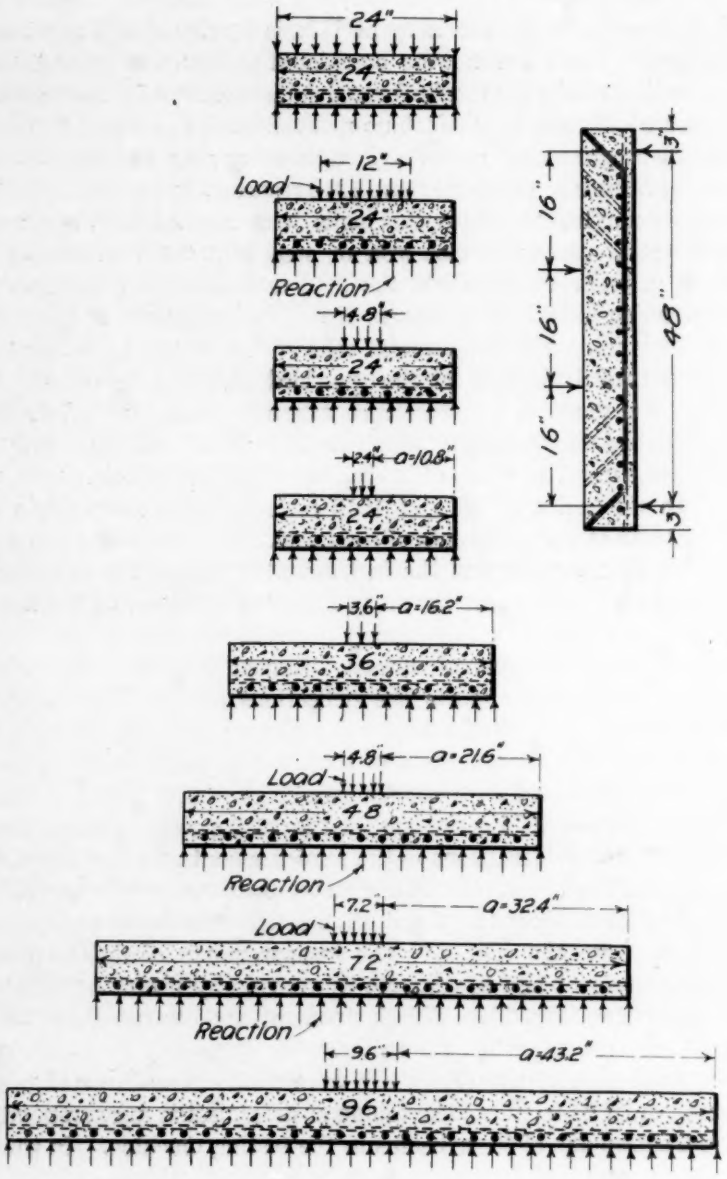

FIG. 1.—WIDTHS OF LOAD FOR BEAMS OF VARIOUS WIDTHS.

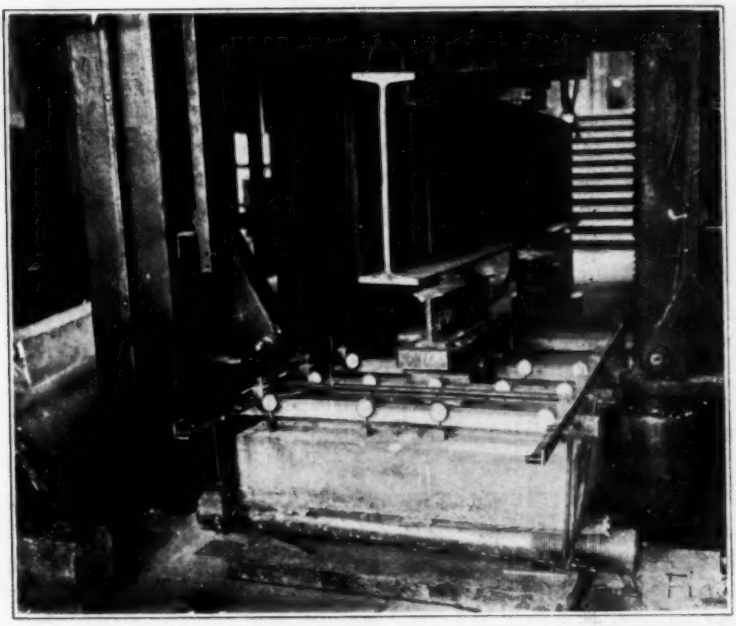

FIG. 2.—WIDE BEAM IN TESTING MACHINE: DEFLECTION INSTRUMENTS IN PLACE; SERIES OF 1910.

FIG. 3.—THE BERRY EXTENSOMETER.

tests the supports consisted of 7-in. and 8-in. I-beams with the I-beams embedded in plaster of Paris at the supports to afford a continuous bearing. The widest beam of this series tested in this way was 36 inches, and the indications are that the deflection of the supporting I-beams was negligible. Two beams in this series were quite different from any others which have yet been tested. They were 96 inches wide and had a span of 30 inches. The supporting I-beams were put in place when the beams were made, and the concrete was poured around one flange and a portion of the web. This is shown in Fig. 5, a photograph of the testing of one of these beams, and in Fig. 8, a photograph of the beam after testing.

Many of the beams of the 1912 series were wider than could be tested in the testing machines which had been used for the previous tests and the apparatus shown in Fig. 6 was used. The load was applied by means of a hydraulic jack and measured by the nickel steel spring shown in this figure. This spring consists of two nickel steel beams each 2 in. deep, 7 in. wide and 17 in. between centers of supports. The loads were applied at points 9 inches apart and the sum of the deflections of the two beams at the center of the span was measured to about $\pm .0001$ in. by means of an Ames gauge-head micrometer set between the beams. The whole weighing apparatus was bolted together firmly so as to form a permanent piece of equipment that could be moved about without impairing its adjustment. Previous to using it in a test it was loaded about ten times sufficiently to stress the steel to approximately 40,000 lb. per sq. in. A deflection of .001 inch at the center corresponds to a load of about 1750 pounds. This method of measuring loads was devised and first applied by Mr. M. L. Enger, Assistant Professor of Theoretical and Applied Mechanics at the University of Illinois. The width of most of these beams was so great as to render impracticable the testing of them in an inverted position. Even in the upright position some difficulty was found in securing sufficient rigidity of support to give a satisfactory distribution of the reaction, at the same time leaving the space under the beams sufficiently open for the taking of observations.

In all the beams of the 1910 series of tests the longitudinal reinforcement was about 1.0 per cent. Some of these beams had

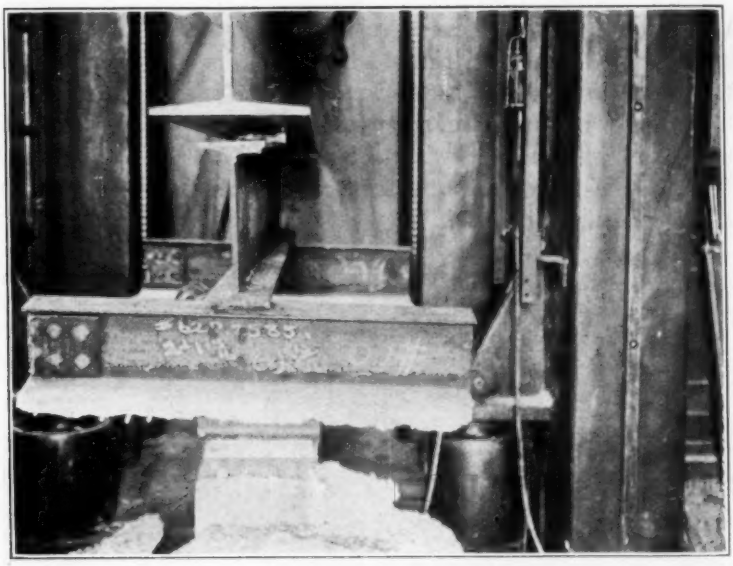

FIG. 4.—WIDE BEAM IN INVERTED POSITION IN TESTING MACHINE; SERIES OF 1911.

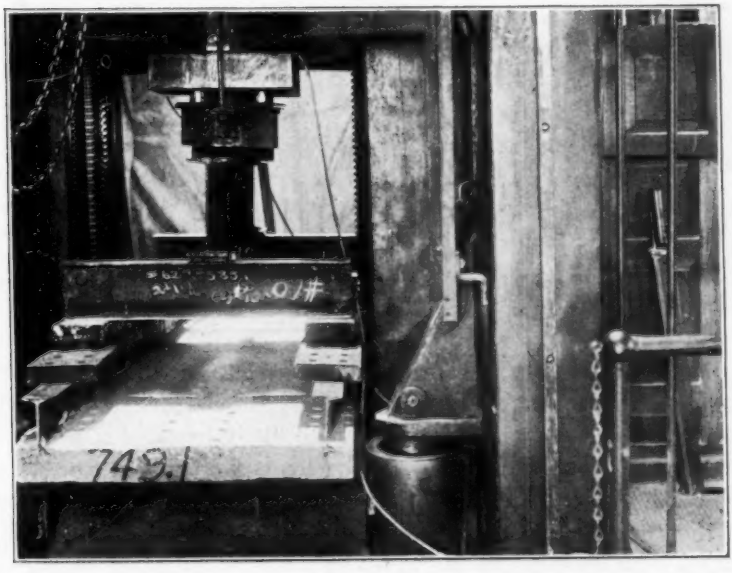

FIG. 5.—TESTING OF 96-IN. BEAM ON SPAN OF 30 INCHES; SERIES OF 1911

0.22 per cent of transverse reinforcement and some had 0.49 per cent, but most of them had none. The depths used were 3 in. and 6 in. and the span of most of the specimens was 48 in. but in a few cases a 72-in. span was used. The beams were 24 in. and 36 in. wide. The width of the load was 0.5, 0.2 and 0.1 times the width of beam, respectively. Most of the attention was placed upon determining what width of load was critical in

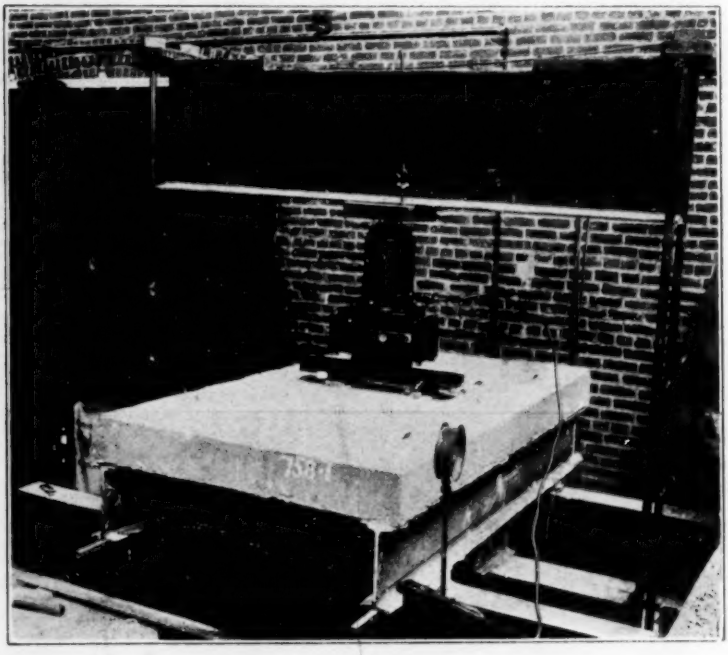

FIG. 6.—TESTING OF 96-IN. BEAM IN HYDRAULIC JACK TESTING MACHINE; SERIES OF 1912.

causing slab failure as defined under *Phenomena of Tests.* It was found that for beams of these widths only the narrowest load used, viz., one-tenth of the width, was at all likely to cause slab failure and the following year most of the loads were concentrated on one-tenth of the beam width, the same type of beam being used. In the 1912 tests the span was still kept at 48 in. and the width varied from 24 in. to 96 in. Two values of longitudinal reinforcement were used, 1.0 per cent and 1.5 per cent. The transverse

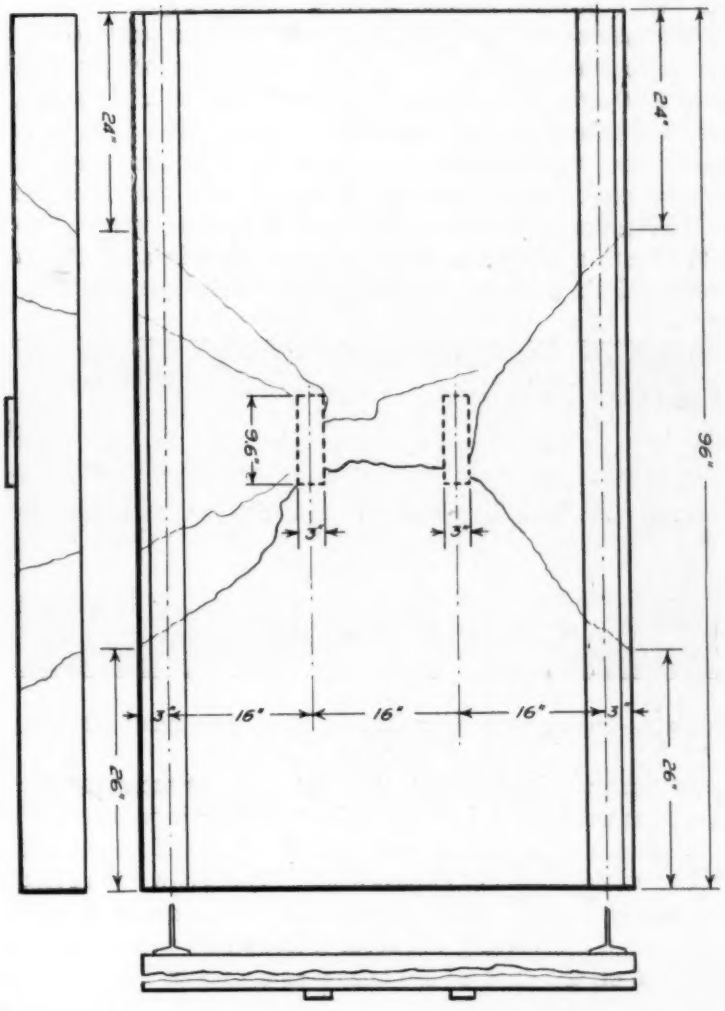

FIG. 7.—CRACKS WHICH ACCOMPANY SLAB FAILURE.

reinforcement was 0.23 per cent, 0.46 per cent and 0.75 per cent respectively. To obviate diagonal tension failure, longitudinal rods were bent up at various places as shown in Fig. 1.

Phenomena of Tests.—In some of the tests a tendency of the corners to curl up has been noted. This was more pronounced in the 1910 series of tests than in any since that time. It is thought this may have been due partially to the fact that most of these beams were cushioned by means of pieces of rubber hose placed between the beam and the rigid support. The natural tendency for the reaction to be concentrated near the central part of the end of the beam as shown in Fig. 1, would result in a greater compression of the hose at this point and a consequent curling up

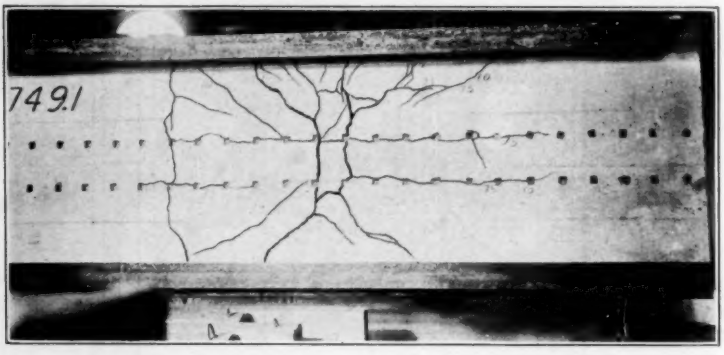

FIG. 8.—CRACKS ON BOTTOM OF VERY WIDE BEAM AFTER FAILURE.

of the corners. In the 1911 and 1912 tests no measurements of deflections were made, but the embedment was in plaster of Paris in all cases, and a marked tendency to curl would have disclosed cracking of the plaster of Paris cushion. This was seldom observed except at loads which gave high stresses in the central longitudinal rods. When this curling did occur it frequently was accompanied by cracks on the ends of the beams as shown in Fig. 7. These cracks were the continuation of a longitudinal crack which formed on the bottom, extending from load point to load point, there forking and extending to the end of the beam at distances from the end varying with the width of the beam. This crack is sketched in Fig. 7. Fig. 8 is a photograph showing characteristic cracks on the tension side of a very wide beam.

In these tests, failure has occurred in four different ways, (1) tension in longitudinal reinforcement, (2) diagonal tension, (3) bond, and (4) transverse tension or failure as a slab. The first three manners of failure are familiar and need no comment, but the fourth is peculiar to this form of test piece and method of loading, hence needs definition. When the tendency of the central element of the beam to deflect more than the elements at the edges, (and hence to cause curvature in a vertical plane at right angles to the direction of the span) is sufficient to cause failure, this will be known as transverse tension failure or as slab failure. In beams having no transverse reinforcement such failure usually follows very closely upon the formation of the longitudinal cracks shown in Figs. 7 and 8. In beams having transverse reinforcement the formation of the longitudinal crack is not necessarily followed by failure as a slab. If the transverse reinforcement passes its yield point before the longitudinal reinforcement does, or before slip of bars occurs, failure is attributed to transverse tension. This of course can be exactly determined only by extensometer measurements. Although punching shear has not been mentioned as a primary cause of failure, diagonal tension caused by it often appeared as a secondary phenomenon and may have been of importance in producing bond failure. When present, the fracture due to this cause usually formed an angle of about 45 deg. with the vertical and sloped away from the loading blocks on all four sides.

Analysis.—The following is given not as exact analysis but rather as an illustration showing the nature of the problems involved.

Fig. 9 represents a wide beam of span s and width b supported on I-beams which may deflect, and loaded uniformly on the longitudinal element along the center-line of the beam. Assume that the deflection at the center of the supporting I-beam f will throw just enough of the pressure from the test piece toward the ends of f to produce a uniform bearing on the test piece as shown in this figure. The reactions as shown will cause a moment in a transverse direction and one in a longitudinal direction. Calling the moment at the center of the span in the longitudinal direction M_L and that at the center of the width in the transverse direction

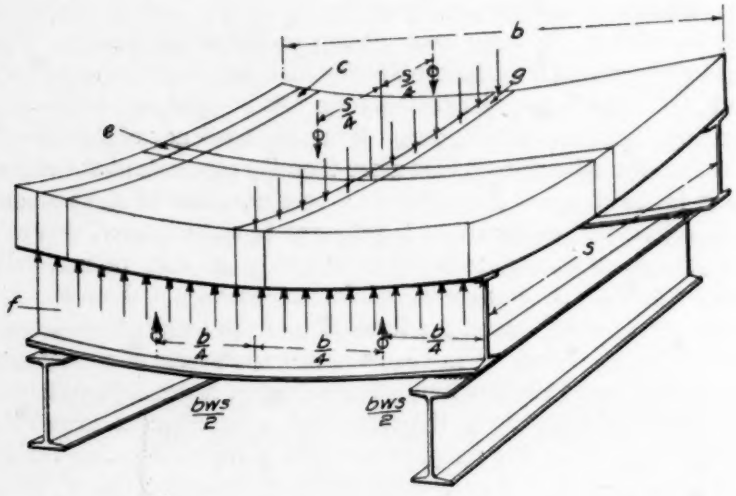

FIG. 9.—SKETCH OF WIDE BEAM WITH LOAD APPLIED UPON CENTRAL ELEMENT.

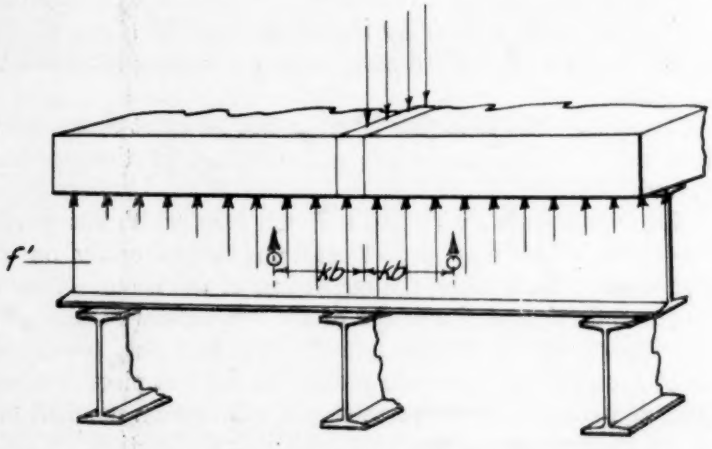

FIG. 10.—SKETCH SHOWING ASSUMED DISTRIBUTION OF REACTION.

M_T and w the load per unit of area of the entire beam, the following relations are derived:

$$M_L = \tfrac{1}{2} w b s \frac{s}{4} = \frac{w b s^2}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$M_T = \tfrac{1}{2} w b s \frac{b}{4} = \frac{w b^2 s}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

The intensity of the transverse moment is greater, across the center of the longitudinal span than near the support, but there is nothing in this analysis to bring out this fact.

For a square beam $b=s$ and $M_T=M_L$, and the same total amount of transverse reinforcement would be required as of longitudinal. However, it is apparent that the load would never be distributed in this way. This case was assumed not because it was thought to be representative of what one would expect to find in practice or even in tests, but to show by a concrete illustration the character of what is meant by transverse beam action, and to choose that illustration at one end of the extreme possible range in conditions. A uniformly distributed reaction would never occur in practice and in tests it could be brought about only by special design. The reaction probably would be somewhat concentrated near the center of the line of support as shown in Fig. 10 and the center of gravity of that portion of a reaction coming upon either half of a support would be at a distance kb from the center of the I-beam f, in which k is likely to be less than $\frac{1}{4}$. Then M_T might be expected to vary from the value given in equation (2) in proportion as k varies from $\frac{1}{4}$. It most likely would always be less than the amount shown by the equation. If now the width of the loaded strip becomes finite the load causing transverse moment will no longer be wbs but will be a function of the overhang, a (the distance from the edge of the loading block to the edge of the beam). The distance to the center of gravity of this load will also be a function of a. A consideration of the data in hand and of various possible distributions of the load causing transverse moment indicates that this load and the distance to its center of gravity may without much error be considered to be proportional to a; then

$$M_T = k_1 w a s \cdot k_2 a = k_1 k_2 w a^2 s = K w a^2 s \quad \ldots\ldots\ldots\ldots\ldots \quad (3)$$

where $k_1 k_2$ and K are constants.

II. RESULTS OF INVESTIGATION.

Beams without Transverse Reinforcement.—In the beams tested, the stress in the longitudinal reinforcement lying along the center line of the beam was much greater than that in the reinforcement near the edge for beams having a width of 48 in. or more, but for widths less than this the stresses were nearly equal across the width of the beam or very irregular in their variation. Figs. 11 to 18 give the load-stress curves and curves showing the variation in longitudinal stress across the width of beams for 24, 36, 48 and 72-in. beams having no transverse reinforcement. In the latter diagrams, termed stress-distribution diagrams, any plotted point indicates the stress measured in the longitudinal rod which is shown in the beam elevation as lying directly under the plotted point.

It will be noted that the shape of the stress curves corresponds to some extent with the reaction curve indicated in Fig. 10 and there is some justification for the assumption that the stress-distribution curves of Figs. 11, 13, 15 and 17 may be used as load curves of a beam whose span is in a transverse direction and whose reaction begins at the longitudinal section through the edge of the loading blocks. The basis for this assumption is here given. In the tests the load was applied at the one-third point of the span upon a central element of the beam. If all the longitudinal elements had their loads applied at the one-third points of the span the loads carried by the various strips would be exactly proportional to the stress at the center of the span. However, it is clear that the load will not travel directly outward so as to be applied at the one-third points of the span of each strip. In fact it is likely that the shear between the edge of the element under the loading block and the element adjacent to it modifies the distribution of load in both elements, giving it more nearly the effect of a uniformly applied load than it would otherwise have. If the variation in load distribution should be from third point loading on the central element to uniform distribution on the element at the edge of the beam, the variation in proportionality of loads to stresses would be 25 per cent and it seems probable that at least up to the time of tension failure in the concrete along a longitudinal line, the variation in load distribution is less than that indicated. If, then, the assumption be used

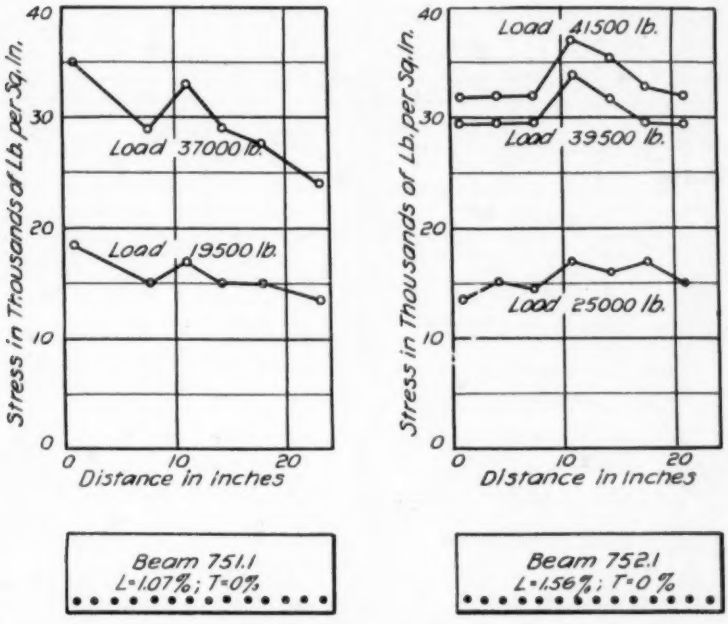

FIG. 11.—STRESS DISTRIBUTION DIAGRAMS FOR 24-IN. BEAMS WITHOUT REINFORCEMENT.

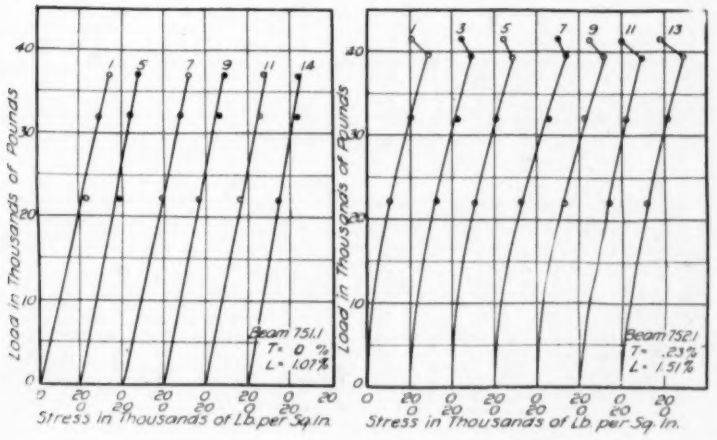

FIG. 12.—LOAD-STRESS DIAGRAMS FOR 24-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

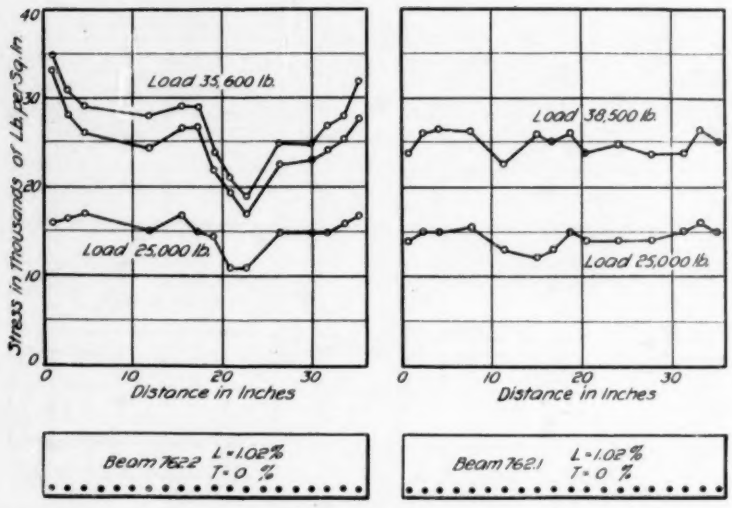
Stress in Thousands of Lb. per Sq. In.
Load 35,600 lb.
Load 25,000 lb.
Distance in Inches
Load 38,500 lb.
Load 25,000 lb.
Distance in Inches
Beam 762.2 L = 1.02% T = 0 %
Beam 762.1 L = 1.02% T = 0 %

FIG. 13.—STRESS DISTRIBUTION DIAGRAMS FOR 36-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

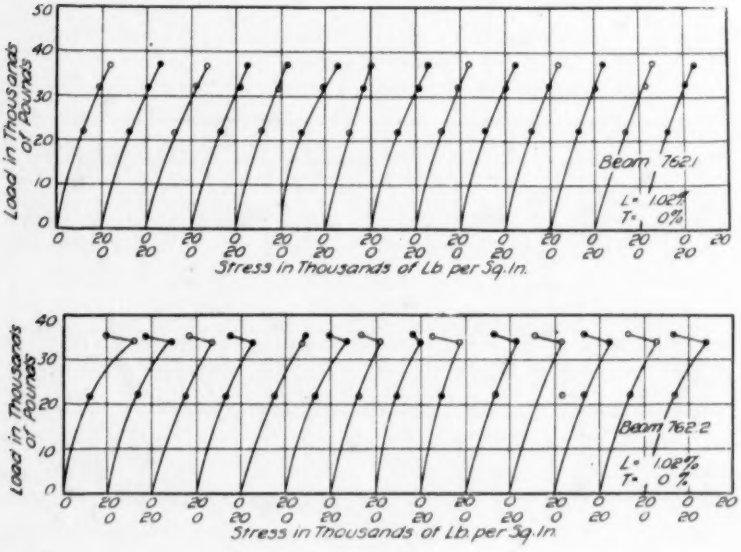
Load in Thousands of Pounds
Beam 762.1
L = 1.02%
T = 0%
Stress in Thousands of Lb. per Sq. In.
Load in Thousands of Pounds
Beam 762.2
L = 1.02%
T = 0 %
Stress in Thousands of Lb. per Sq. In.

FIG. 14.—LOAD-STRESS DIAGRAMS FOR 36-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

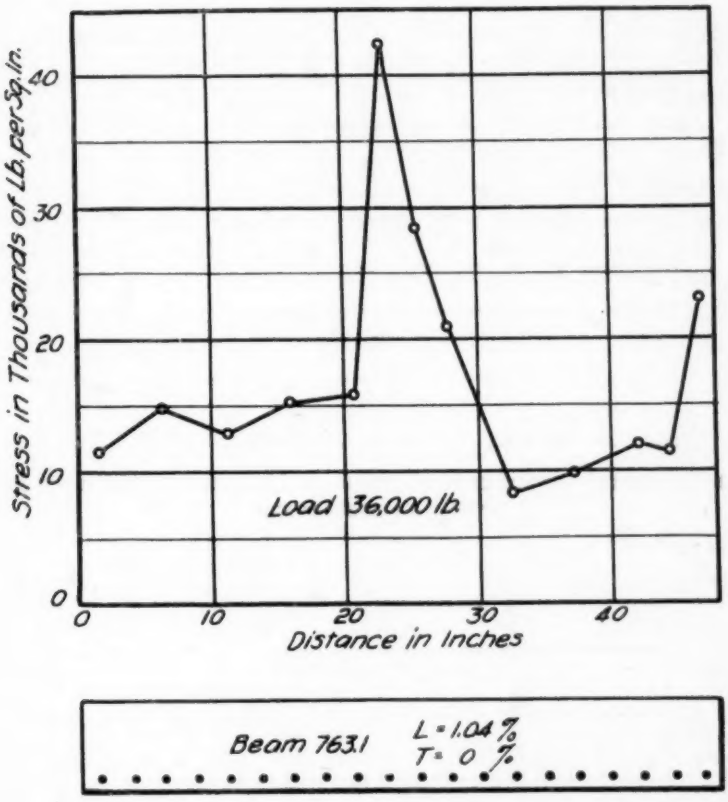

FIG. 15.—STRESS DISTRIBUTION DIAGRAMS FOR 48-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

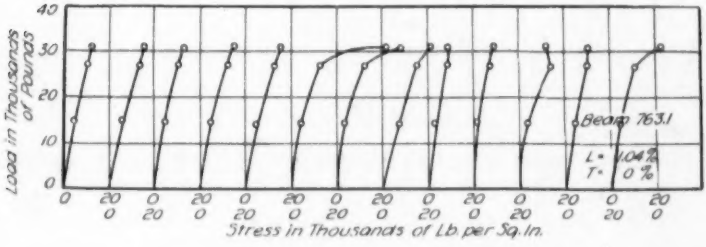

FIG. 16.—LOAD-STRESS DIAGRAMS FOR 48-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

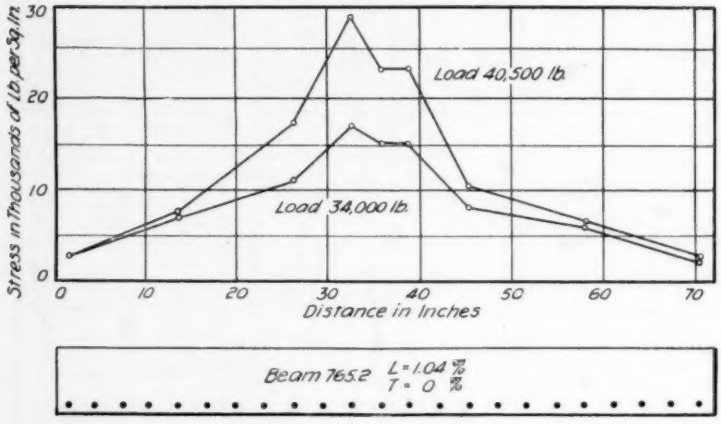

FIG. 17.—STRESS DISTRIBUTION DIAGRAMS FOR 72-IN. BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

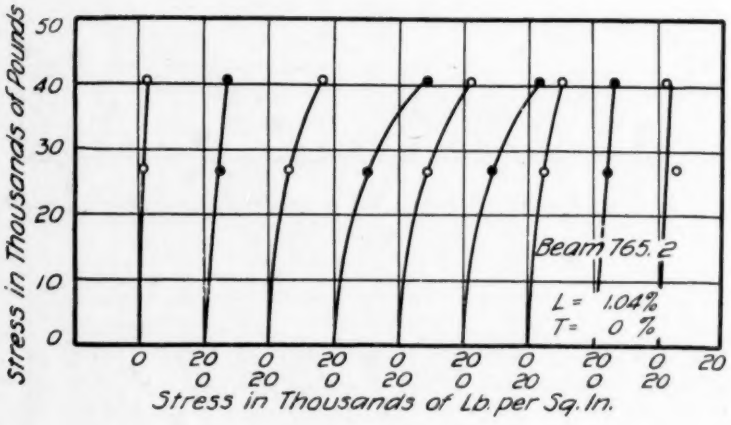

FIG. 18.—LOAD STRESS DIAGRAMS FOR 72-IN. TRANSVERSE BEAM WITHOUT REINFORCEMENT.

that the loads on the longitudinal elements are proportional to the measured stresses in them the transverse moments calculated for the longitudinal section along the edge of the loading blocks for a load of 1333 lb. per sq. ft. on all the beams considered under the present head are as shown in Fig. 19. It may be seen from equation (3), that if M_T be plotted as ordinates of a curve in which the abscissas are values of overhang the curve will be a parabola if the form of the equations is correct. A parabola which fits the results fairly well is shown by the dotted line in Fig. 19. In Fig. 20 the distances, X, to the center of gravity of the assumed load curves are plotted as ordinates and the width of beams as abscissas. In this figure the ratios of ordinates to

TABLE I.—MODULUS OF RUPTURE FOR BEAMS WITH NO TRANSVERSE REINFORCEMENT, LB. PER SQ. IN.

	No. of Tests.	Average.	Maximum Variation from Average, per cent.
24-inch beams	3	253	34
36-inch beams	1	316	..
48-inch beams	4	365	26
72-inch beams	2	371	1

corresponding abscissas will be values of k. For a constant k the curve would be a straight line. The straight line shown in the figure gives the value of ka which if multiplied by a constant ratio of the load on the beam will give the parabola. The curve through the plotted points indicates that k decreases as b increases. The straight line gives 0.23 as a mean value of k for the beams with no transverse reinforcement. If for the maximum load carried, the transverse moment at a longitudinal section along the edge of the loading blocks be calculated by the method outlined above the values of the modulus of rupture obtained are as given in Table 1. In this table the effective width is taken as the full length of the beam, and the values given are averages.

The variation in values for similar test pieces is in some cases large but not larger than in other kinds of test pieces where there is so much chance for eccentricity in loading. The agreement

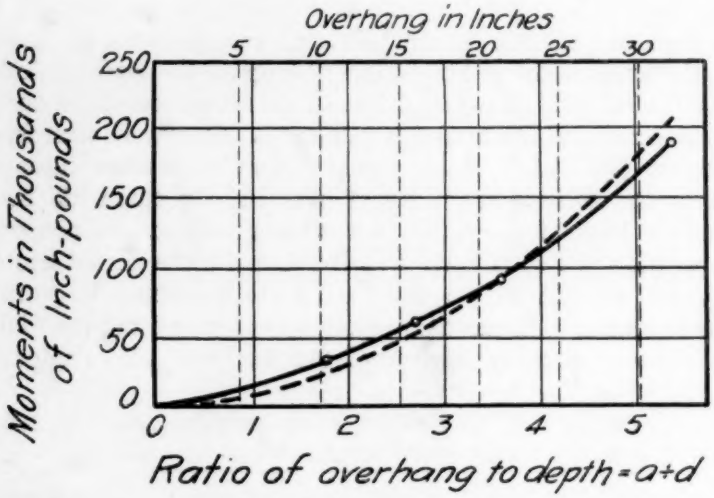

FIG. 19.—TRANSVERSE MOMENTS IN BEAMS WITHOUT TRANSVERSE REINFORCEMENT.

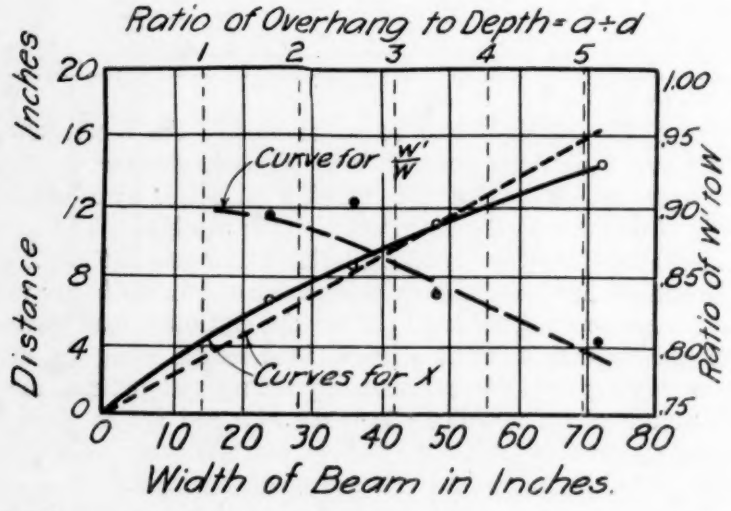

FIG. 20.—RELATIONS BETWEEN WIDTHS OF BEAMS AND LOAD DISTRIBUTION.

X curves: distance from edge of loading blocks to center of gravity of load outside of load blocks. $W' \div W$ curves: ratio of load outside of loading blocks to total load on beam.

between values for different widths is sufficient to indicate that there is some basis for the method used.

The actual value of the modulus of rupture as calculated is not far from that which we might expect to obtain. The values for the 24-in. and 36-in. beams are low. As noted below, the extent to which slab action affected these results is uncertain. It is true that at the load causing failure in these narrower beams a high tensile stress in the steel was present, and except for this weakness it is possible that they would have developed as high a modulus of rupture in the transverse direction as did the wider beams. The use made here of the data is allowable only on the basis that failure was by transverse tension in the concrete. This was true in all the 72-in. and 48-in. beams having no transverse reinforcement. In the 36-in. and the 24-in. beams there is some room for question, but in even the 24-in. beams the variation in stress across the width of the beam shown in Fig. 12 indicates the probability that although in beam 752.1 a high average steel stress was developed, failure as a slab may have been imminent.

Transverse Moments in Beams Having Transverse Reinforcement.—An effort was made to apply to the beams having transverse reinforcement the same methods of calculating transverse moments as has been used in the beams having no transverse steel. That is, the loads carried by longitudinal elements were assumed to be proportional to the steel stress in the element. On this basis the transverse moments in beams having 0.23 per cent transverse steel (Figs. 21 to 26 inclusive) were calculated for a section at the edge of the loading block. Using the moments thus calculated to determine stresses in the transverse steel, results were obtained which are far beyond the ultimate strength of the steel. The actual resisting moment at this section was then calculated by taking the summation of the products of the measured stresses, the steel areas and the arm of the resisting couple (assumed to be about 0.87 of the depth to the transverse steel). The values thus obtained for a load of 1875 pounds per square foot plotted against values of the ratio $\frac{a}{d}$ form the lower curve in Fig. 27. The points on the upper curve are obtained by the method previously described and are uniformly about four times as large as those of the lower curve. Apparently the methods

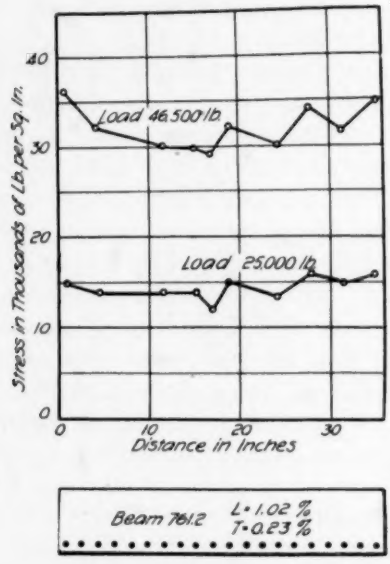

FIG. 21.—STRESS DISTRIBUTION DIAGRAM FOR 36-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

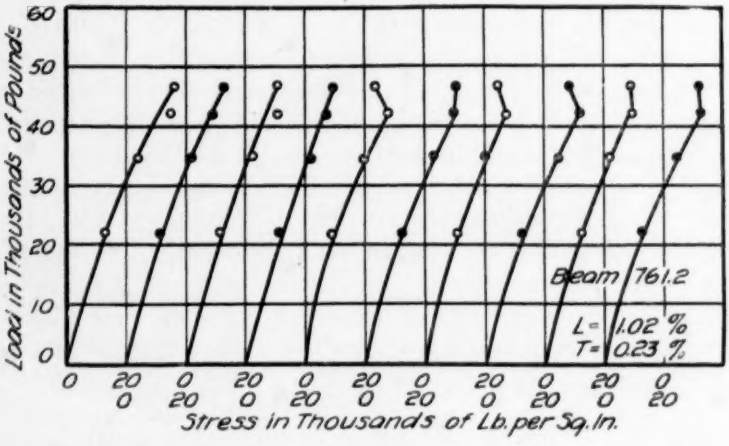

FIG. 22.—LOAD-STRESS DIAGRAM FOR 36-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

which give fairly consistent results for the beams having no transverse reinforcement are not applicable where transverse steel is present. Probably when the concrete fails in tension along a longitudinal section the stiffness of the test piece as a transverse beam becomes much less, and a much larger part of the reaction is concentrated near the center of the width of the beam. This

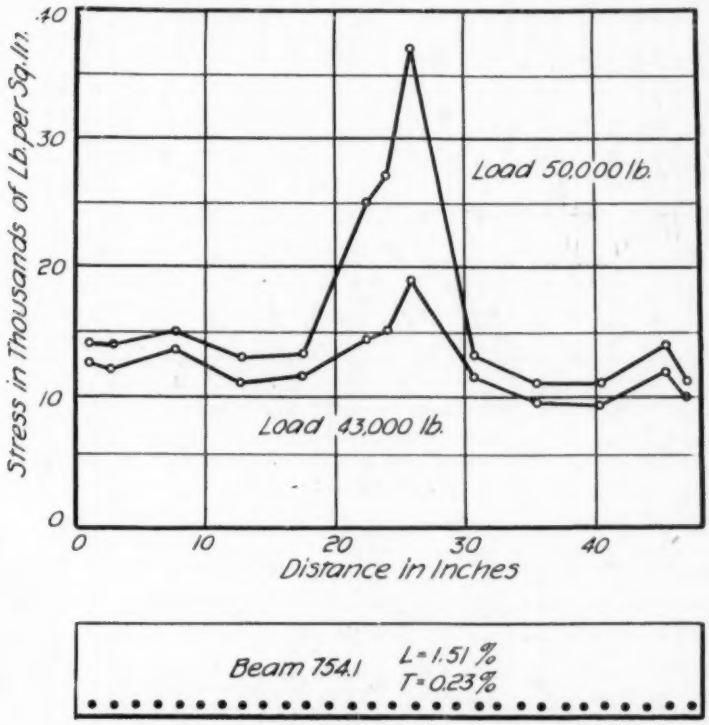

FIG. 23.—STRESS DISTRIBUTION DIAGRAM FOR 48-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

would make the transverse moment much smaller in proportion to the load on the beam than for beams with no transverse reinforcement. There are other phenomena which apparently have a bearing upon this matter but which are somewhat conflicting and too complicated to be discussed in a short space.

Effect of Variation in Amount of Longitudinal Reinforcement.—

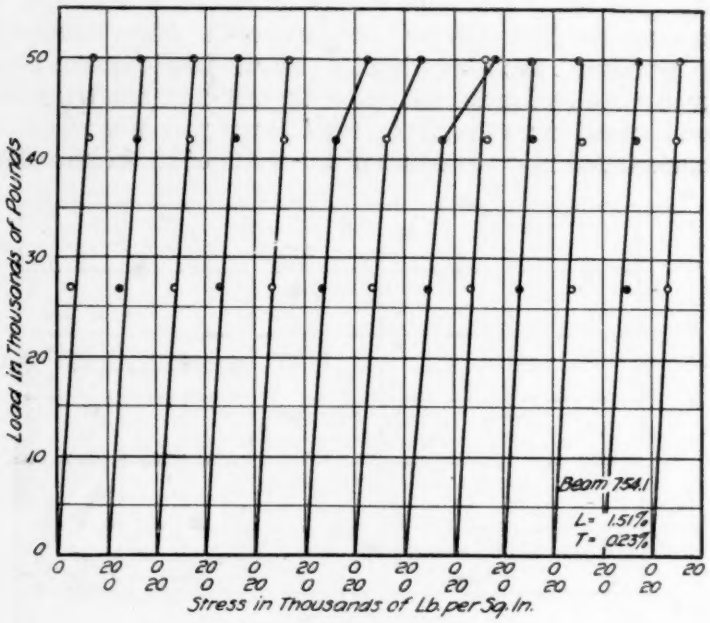

FIG. 24.—LOAD-STRESS DIAGRAM FOR 48-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

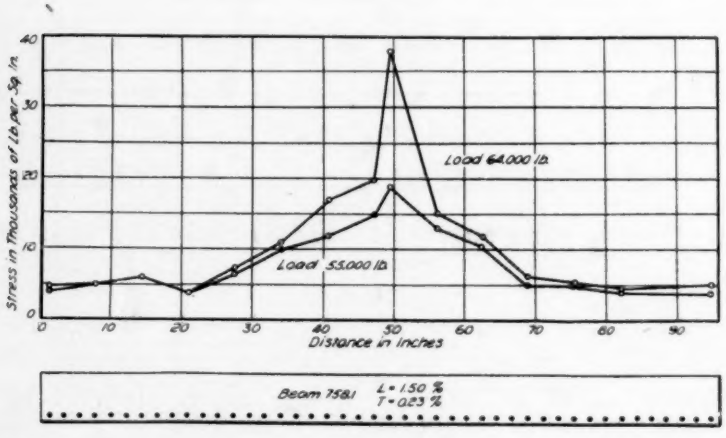

FIG. 25.—STRESS DISTRIBUTION DIAGRAM FOR 96-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

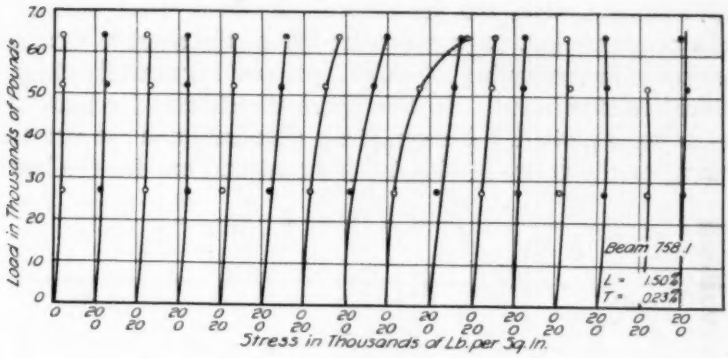

FIG. 26.—LOAD-STRESS DIAGRAM FOR 96-IN. BEAM WITH TRANSVERSE REINFORCEMENT.

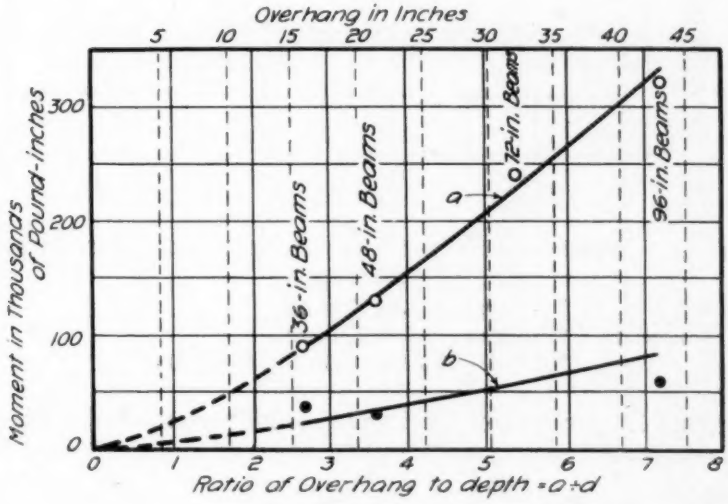

FIG. 27.—TRANSVERSE MOMENTS IN BEAMS HAVING TRANSVERSE REINFORCEMENT.

Note: Curve *a* assumes load on transverse beam proportional to stress in longitudinal steel. Curve *b* is summation of products of transverse steel stresses and arm of resisting couple.

To show the effect of a variation in the per cent of longitudinal reinforcement when the per cent of transverse reinforcement is kept constant, Fig. 28 was prepared from the average measured stresses in beams having 1.0 and 1.5 per cent respectively of longitudinal reinforcement. In this figure the stresses for individual beams were plotted. A mean stress distribution curve was then drawn for the beams having 1.0 per cent and one for the beams

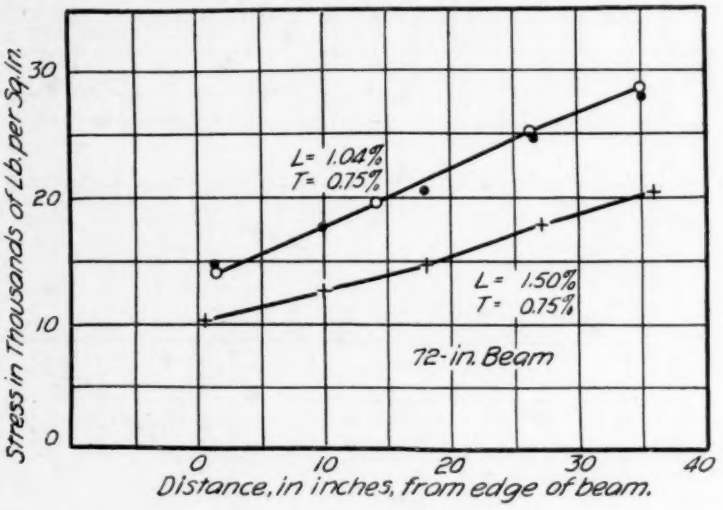

FIG. 28.—DIAGRAM SHOWING EFFECT OF VARIATION IN LONGITUDINAL REINFORCEMENT.

Legend
- \+ Stresses in 1.50 % beams.
- o Stresses in 1.04 % beams.
- • 1.50/1.04 x stresses in 1.04 % beams.

having 1.5 per cent of longitudinal reinforcement. Multiplying the stresses shown in the 1.5 per cent curve by the ratios of the two percentages $\left(\frac{1.5}{1.0}\right)$ the resulting values agree closely with those of the 1.0 per cent curve. This relation is of course to be expected with narrow beams. Its significance here is that variation in longitudinal reinforcement affects only the unit stress at any part of the beam but does not affect the total stress at that point. In other words, the distribution of stresses seems not to be affected by variation in amount of longitudinal reinforcement.

Effect of Variation in Amount of Transverse Reinforcement.—The effect of increasing the amount of the transverse reinforcement while the amount of longitudinal reinforcement is kept constant is towards making the stresses in the longitudinal rein-

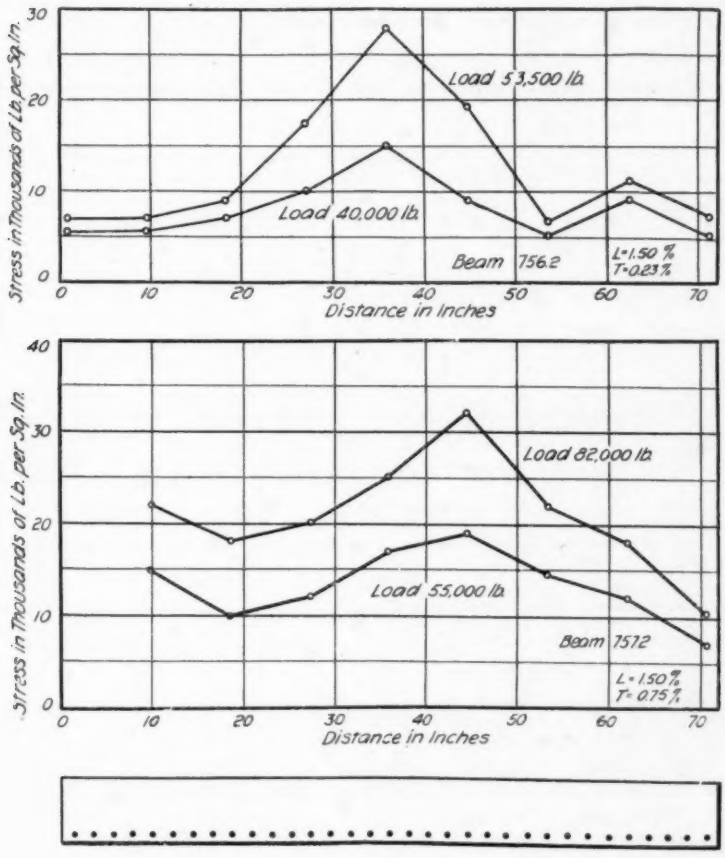

FIG. 29.—STRESS DISTRIBUTION DIAGRAMS SHOWING EFFECT OF VARIATION ON AMOUNT OF TRANSVERSE REINFORCEMENT.

forcement equal throughout the width of the beam at a given load. This is shown in Fig. 29 by a comparison of the upper curve for beam 756.2 (load 53,500 lb.) with the lower curve for beam 757.2 (load 55,000 lb.). No expression for relation between the amount

of transverse reinforcement and its effect in equalizing the stress in the longitudinal steel has been determined.

Stress in Transverse Reinforcement.—In Fig. 30 the stress at various positions on a transverse reinforcing bar are plotted as ordinates and distance from the center of the beam as abscissas.

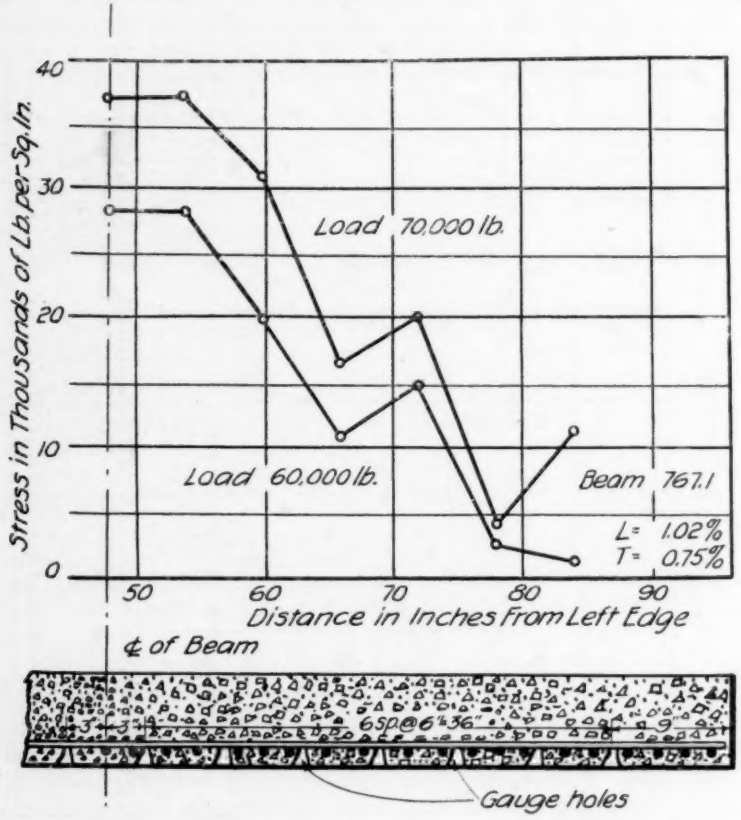

FIG. 30.—DIAGRAM SHOWING STRESSES IN TRANSVERSE BAR.

This curve indicates that the stress reaches a maximum at about the edge of the loading block, as was assumed in the calculation of the modulus of rupture for the beams having no transverse reinforcement. It was thought that by means of the stress curves for transverse rods and for the concrete above it in a transverse

direction the moment curves and hence the distribution of the load producing the transverse stresses might be obtained, but there are indications of such complications in the action as not to permit of so simple a solution.

Deformations in Concrete.—In a few of the tests deformations in the concrete were measured in both the longitudinal and the transverse directions. The stress distribution in the longitudinal direction appears to be much the same as for the reinforcement.

TABLE II.—COMPARISON OF CALCULATED STRESS IN LONGITUDINAL REINFORCEMENT WITH AVERAGE OF THE MEASURED STRESSES.

Beam No.	Width, in.	Per cent Reinforcement.		Load Considered, lb.	Measured Stress, lb. per sq. in.		Calculated Stress, lb. per sq. in.
		Longitudinal.	Transverse.		Maximum.	Average.	
750.1	24	1.07	0	12,000	38,500	34,500	48,900
751.1	24	1.07	0	37,000	35,000	29,600	37,700
752.1	24	1.56	0	41,500	37,000	33,600	29,800
762.1	36	1.02	0	38,500	26,500	25,300	27,300
763.1	48	1.04	0	31,600	42,000	15,630	16,500
763.2	48	1.09	0	40,200	42,000	21,400	20,200
753.1	48	1.51	0	42,500	21,000	12,300	15,900
753.2	48	1.51	0	42,000	22,500	15,570	15,560
765.1	72	1.04	0	32,000	20,000	10,300	11,100
765.2	72	1.04	0	40,500	29,000	10,960	14,100
761.1	36	1.02	.23	38,600	34,000	30,100	27,500
761.2	36	1.02	.23	46,500	35,000	33,000	33,100
754.1	48	1.51	.23	50,000	37,000	16,230	18,530
754.2	48	1.51	.23	57,000	36,500	21,300	21,100
764.1	48	1.04	.78	66,000	42,000	35,800	34,300
764.2	48	1.04	.75	65,500	38,000	36,200	34,100
756.1	72	1.50	.23	67,000	29,000	13,620	16,600
756.2	72	1.50	.23	53,500	27,700	12,920	13,260
766.1	72	1.04	.75	80,700	42,000	25,000	28,000
766.2	72	1.04	.75	61,500	33,000	26,000	21,300
757.1	72	1.50	.78	76,500	31,000	19,700	18,950
757.2	72	1.50	.75	82,000	32,000	21,500	20,300
758.1	96	1.50	.23	64,000	38,000	9,800	11,920
758.2	96	1.50	.23	59,500	42,000	10,800	11,080
767.2	96	1.02	.75	73,800	39,500	24,000	19,700
767.1	96	1.02	.75	71,500	37,000	17,500	19,090

In the transverse direction the deformations were very small, this being due probably to the large amount of concrete available to take the transverse compression. There was some indication that the stress passed from compression near the edge of the loading blocks to tension nearer the edge of the beam. In all cases where this phenomenon was observed the amount of tension was so very small that more information is needed to establish the fact of its existence. Its meaning, if present, must be that not all transverse strips act as beams and that if the beam as a whole acts as a

transverse beam, other portions than the one under consideration must take an increased amount of compression due to the fact that tension in concrete is present on the strips which pass near the loading block.

Average Measured Reinforcement Stresses.—The use of the ordinary formula $M_L = pfjbd^2$ for calculation of longitudinal steel stresses in wide beams gives results usually much lower than those which exist near the center of the width of the beam. However, it has been found that the average of the measured stresses is in

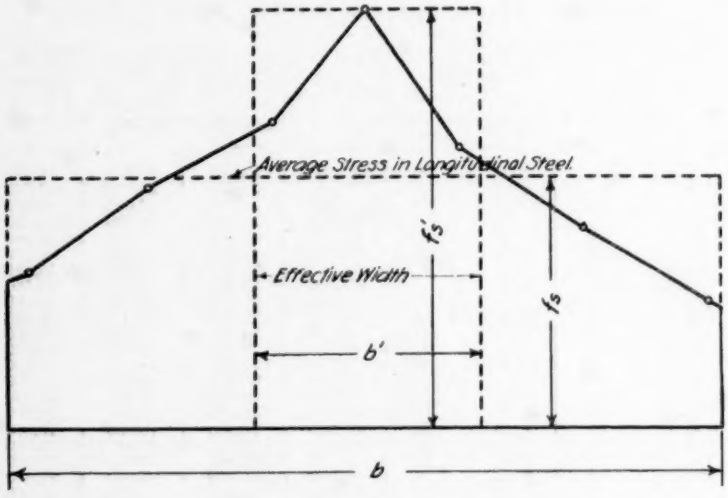

FIG. 31.—DIAGRAM SHOWING RELATION BETWEEN EFFICIENCY AND EFFECTIVE WIDTH.

reasonably close agreement with the stresses calculated for the same load. This is shown in Table II, in which the concrete is assumed to be stressed to 0.5 of its ultimate strength.

Relation between Efficiency and Effective Width.—The total stress in the longitudinal reinforcement is equal to the area times the average stress. But the average stress apparently is approximately equal to the stress calculated by the ordinary methods used for reinforced concrete beams. The total stress for any load can then be calculated. The significance of this fact is shown in the following paragraph.

In Fig. 31 the following notation is used:

f_s = the average stress in the longitudinal steel at the center of the span.

f'_s = the maximum stress in the longitudinal steel.

$\frac{f_s}{f'_s}$ = the efficiency of the beam.

b = the width of the beam.

b' = the effective width of the beam.

Since the maximum stress will govern the design, f'_s will be the designing stress. By effective width is meant the width in which if the stress in all the longitudinal reinforcement be considered to be equal to the maximum, the total stress will be equl to the actual total stress in the beam.

From the above relations it will be seen that the reinforcement stress on a section at the center of the span is represented to some scale by bf_s and to the same scale by $b'f'_s$ provided that the longitudinal steel is uniformly spaced.

$$\therefore\ bf_s = b'f'$$

$$\frac{b'}{b} = \frac{f_s}{f'_s} \text{ and}$$

$$b' = b\frac{f_s}{f'_s}$$

From the data obtained in the tests values of the ratio $\frac{f_s}{f'_s}$ (her termed efficiency) have been calculated and are given in Fig. 32.

The efficiency generally decreases as the load increases and these values are usually those for the maximum load on the beam. Therefore, at the stage when the reinforcement is stressed only to an ordinary working stress the efficiency and the effective width would be larger than is indicated in the diagram. However, it seems that the efficiency at the maximum load is the one to be used in design since the real factor of safety must be based on the ultimate strength of the beam.

Relation between Rigidity of Supports and Stress Distribution.—Such a distribution of reactions as is shown in Fig. 9 along the beam f would be expected to result in a considerable uniformity of the longitudinal stresses at the center of the span and a consequently high efficiency. However, it will result also in high

stresses in the transverse reinforcement and a consequent tendency toward slab failure. The distribution of reaction shown in Fig. 10 will produce smaller transverse stresses but will tend to cause greater stresses in the central longitudinal elements and hence will result in smaller efficiencies. For economical use of material

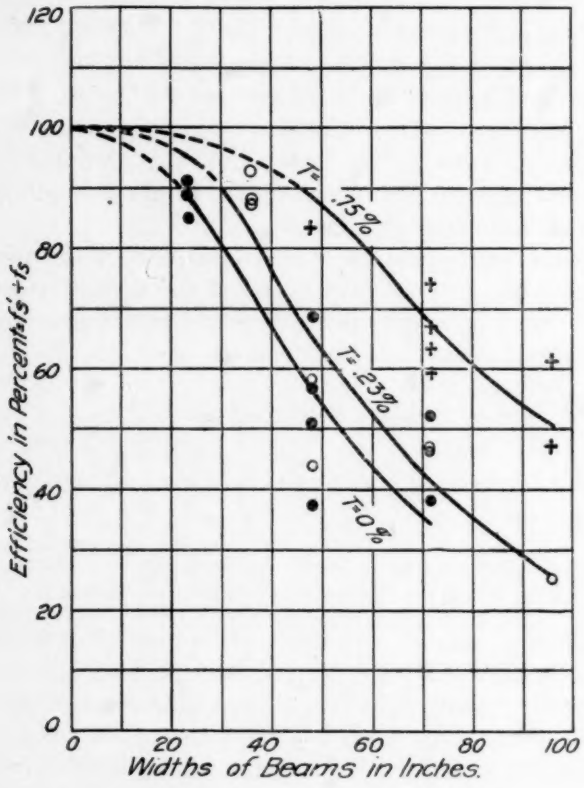

FIG. 32.—DIAGRAM SHOWING RELATION BETWEEN EFFICIENCY AND WIDTH OF BEAM FOR VARIOUS AMOUNTS OF TRANSVERSE REINFORCEMENT.

then it would seem that a balance must be found between too great rigidity of support and no transverse reinforcement, and too great flexibility of support and the large amount of transverse reinforcement which would be required. No attempt is made here to show how this balanced condition is to be found. The stress distribution curve for beam 762.2, Fig. 13, illustrates what

might be expected to result from too great flexibility of supports with too little transverse reinforcement, although it is not known that in this case the result shown was brought about in this way.

The tests which so far have been made have not yielded much data that are directly applicable to the design of beams of such dimensions as would be used in practice, but the determinations of effective widths which have been made may be expected to be of some use as a guide. Some data have been obtained on the variation in effective width due to variation in the depth and in the span of beams but the tests do not cover sufficient range and were not made on a sufficient number of test pieces to afford a basis for conclusions. The method used for calculating transverse bending moment gives reasonable results for the beams having no transverse reinforcement, but its use depends upon a knowledge of the distribution of the longitudinal stresses. Since an expression showing this distribution has not been determined, this method could not be applied to design.

The indications that in wide beams the effective width is not changed by variation in the percentage of longitudinal reinforcement and that the average of the stresses in the longitudinal reinforcement are in close agreement with the stresses calculated by ordinary methods, are of importance in design. Most of the information gained even if not of great practical use indicates progress towards a solution of the problems presented in the design of wide beams. Further tests are projected which should give valuable information.

Before publ. V. 98. 393

SAND AND GRAVEL WASHING PLANTS.

By Raymond W. Dull.*

There has been a very marked change, during the last few years, in preparing materials for concrete purposes. We seldom see the old obsolete method of sending a team out to a river bank, or gravel pit, for a load of gravel, which is dumped in the aggregate, regardless of size, proportion or cleanliness. We now see a modern factory for handling, sizing and washing the materials, which decreases the cost of handling, prepares it for different kinds of uses, and best of all, makes it suitable for the purpose.

Unclean material, with poor workmanship, has been responsible for nearly all failures. A contractor who is not particular about his materials is seldom particular about his workmanship. It is useless to buy high grade cement and mix it with gravel having even a thin film of dirt coating the pebbles, for the cement cannot adhere to the pebbles.

The successful method used in the modern gravel washing plant is to agitate the material until the impurities are in suspension in the water, drawing the water off continually, and then rinse the material as it leaves the screens. It is important to leave the water with the material until the last, in order to get sufficient agitation, and that is why separate screens are used for each size.

Screens.

The writer, after considerable study, developed the hook-bolt type conical washing screen (Fig. 1), which on account of its simplicity, low power, efficiency and durability, has served the purpose exceedingly well.

These screens are set in a series (Fig. 2) with spouts carrying the sized material to succeeding screens below, while the rejections from the screens discharge from the mouth of the bell into the bins below. The material gets its rinsing from fresh water pipes at the mouth of the screens.

The upkeep of this screen is low because the bearings and

* President, Raymond W. Dull and Company, Aurora, Ill.

drives are away from the grit and water. The large hook-bolts which support the screen cone do away with all fastenings to the casting, and the screen cones can be changed in less than an hour's time.

Appreciating the advantage gained if all chain drives could be eliminated, the inclined conical screen was conceived, a still more radical departure in screen construction. All the screens in this case are mounted on one shaft, and a pair of gears drives everything (Fig. 3). This type of screen differs in its operation from the other screen, for the material is delivered into the large end and travels toward the small end, while with the hook-bolt type the material is delivered into the small end and travels toward the large end.

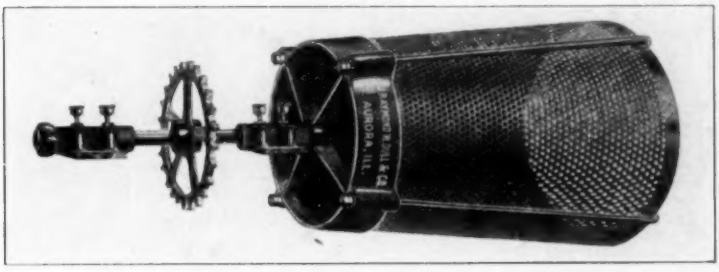

FIG. 1.—CONICAL SCREEN OF HOOK-BOLT TYPE.

The inclined conical screen has the advantage because the large end does the principal work. The large end has more perforations, more wearing surface, and is in fact, equivalent to a larger screen. It therefore has greater efficiency, requires less power and has small upkeep. The screens have longitudinal joints, and can be dismantled from the shaft without disturbing the shaft. Walkways are also provided on both sides of the screens, to make them accessible.

WATER SUPPLY.

The largest quantity of water is introduced when the material is first starting into the screens, and additional fresh water is added at each screen. The water supply is a very important detail of the plant, and sometimes success or failure depends

entirely on this feature. After the material passes through all the screens, the remaining materials are the soil water and the sand.

SETTLING TANK.

A device extensively used for separating the sand from the soil water is the wooden tilting tank of Mr. Charles Stebbins, of Chicago. This tank rocks on two bearings on the sides of the tank. It is divided into two triangular compartments, one for the water and accumulation of sand, and the other compart-

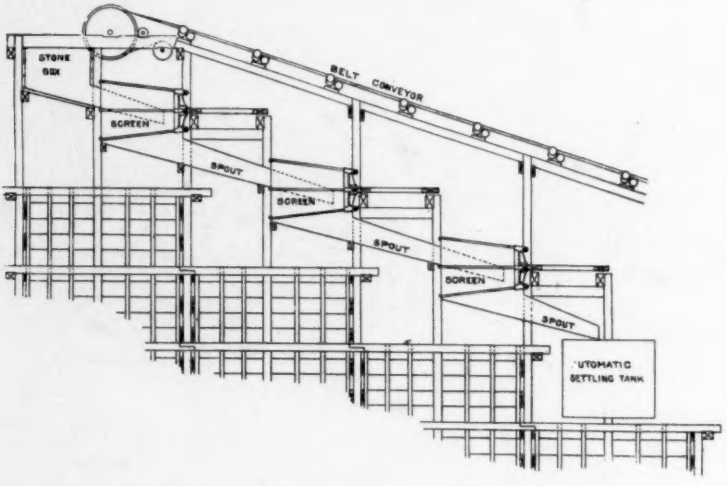

FIG. 2.—ARRANGEMENT OF CONICAL SCREENS.

ment for counterweights, acting counter to the weight of the water and sand.

The valves of the tank consist of pipes, fastened to the tank, with their ends bearing against a fixed timber. When the sand accumulates enough weight, the tank tilts, drawing the pipes from the timber stop, and allowing the sand to escape into the bins below. When a certain amount of sand escapes, the tank tilts back, closing the valves.

A modified form of tilting tank made of steel, which is somewhat more sensitive and designed to permit adjustment of the equilibrium, is shown in Fig. 4. It consists of a triangular shaped

tank, rocking on two knife-edged brackets on the sides, and counterbalanced by a lever and weight. The lever is designed to increase the moment arm of the weight, which retards the tendency to tilt open too far, and does not permit flushing of the soil water into the sand bin below. The valve castings, which have "V" edges, bear against steel plates with rubber faces, which are pivotally connected to timber. The operation of these tanks is very gratifying and results are very satisfactory.

WASHING THE AGGREGATE.

The reason it is so difficult to wash sand and gravel in the aggregate is simply this: When getting rid of the water, it must

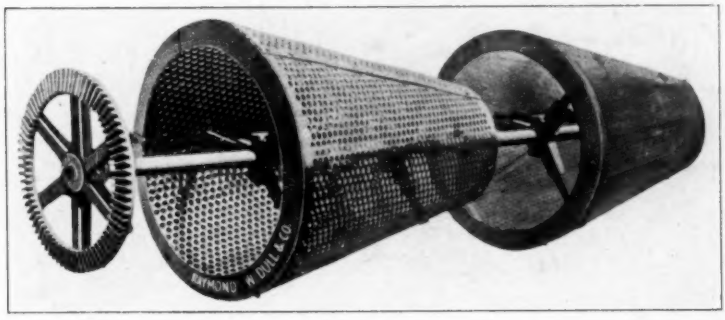

FIG. 3.—INCLINED CONICAL SCREEN.

not be allowed to percolate through the material to drain off. To do this makes a regular filter bed operation, and all the impurities in the water are held in the material.

Of course a constant stream of water can be added, until practically all the impurities are taken away, and then the clear water can be drawn off at the bottom. This means new material must not be added, and it reduces the operation to a batch or intermittent method. The quantity of water necessary also is excessive. There are devices on the market for mechanically lifting the material out of the water, which produce material washed very well, but the power and upkeep of this class of machinery is very great. It is also impossible to get the water from the aggregate mixture by using a tilting tank, because the gravel will not allow the device to work properly.

It is better to wash the material through sizing screens and then mix together as the material comes from the screens. A plant was designed to segregate the material into separate bins, or combine them together in one bin for a contractor, using the material for reinforced concrete work. This plant assembles all the material, when desired, into the central bin. The material from the first screen is not included, on account of its large size. The proportions are regulated by the slide gates, which

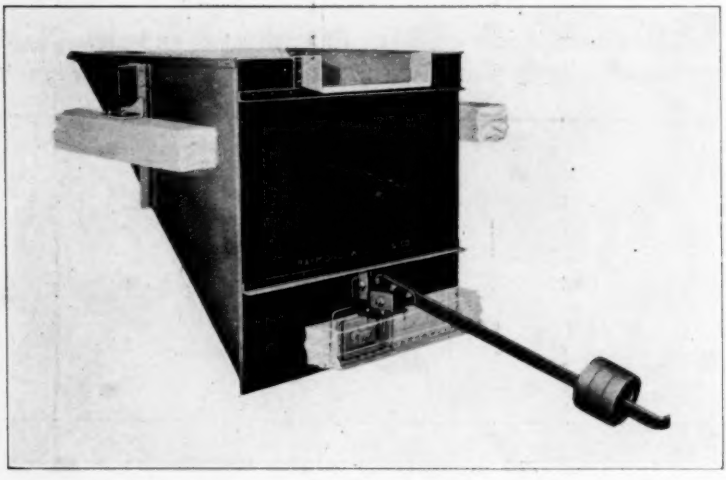

FIG. 4.—TILTING SAND SEPARATOR.

cut off so much of the flow, and shunts the remainder into the other bins.

MIXING.

Great care must be taken, however, in relying on a mixture accumulated in bins, for the mixture of material may be perfect in the bins, but separated to a distressing extent, when the material is drawn from the bin. The coarse material will flow more readily and go to the far side of the car or wagon, and the fine material, which is more sluggish, will drop into the car on the near side. The result is that one side of the car has coarse material and the other side fine material.

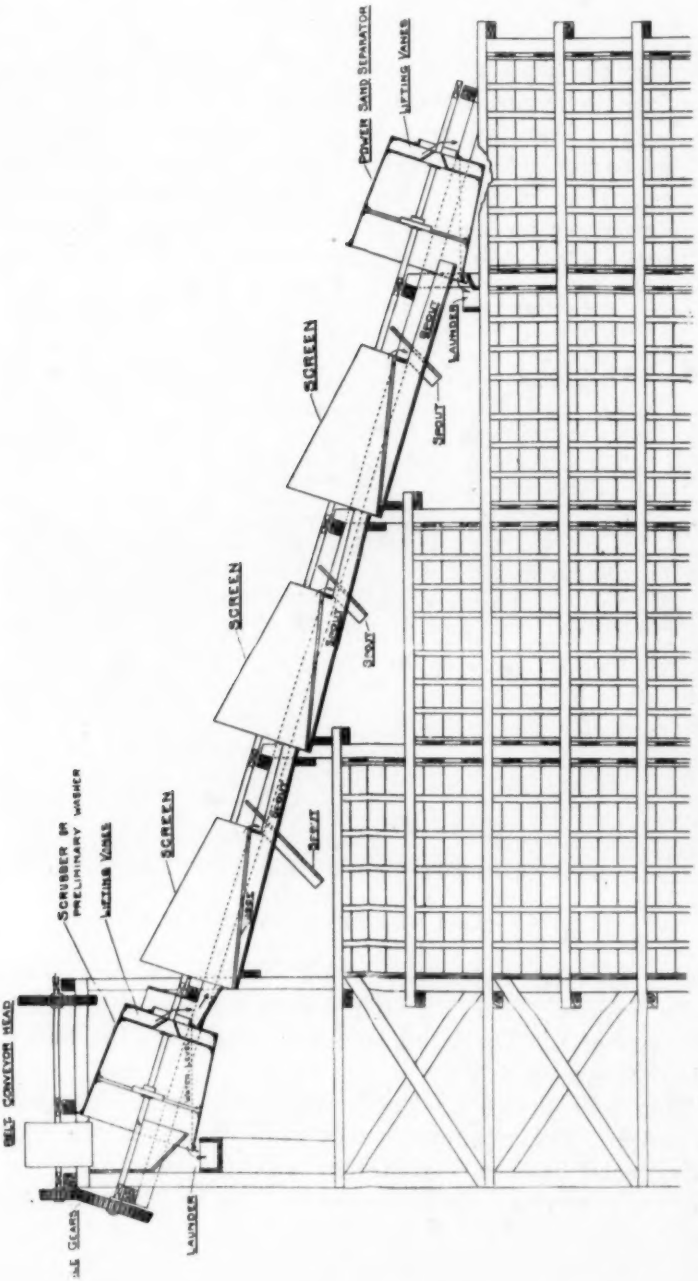

FIG. 5.—PLANT WITH PRELIMINARY SCRUBBER.

SCRUBBER.

So far the washing problems have been attacked from a mechanical standpoint, but the aid from a chemical analysis is appreciated. There are some materials which have impurities very hard to eliminate. Some clays will not succumb to ordinary processes. This led to the introduction of a preliminary scrubber for materials unusually hard to wash, Fig. 5. It consists of a frusto-conical cylinder, shaped in a manner to retain con-

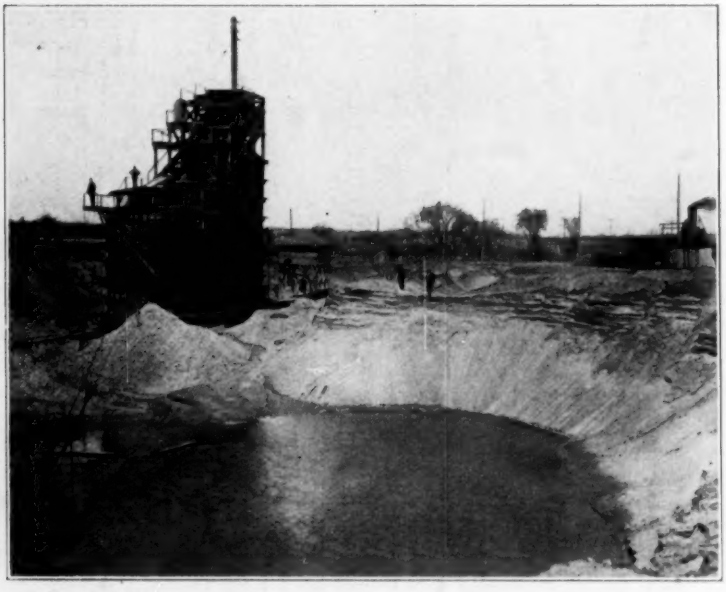

FIG. 6.—PLANT WITH DRAG-LINE EXCAVATOR.

siderable material and water, which tumbles about when the cylinder rotates. This agitation breaks up the clay and the worst part of the impurities is carried off before the material reaches the screens.

This same device used as a mechanical sand separator is also shown. The separator is hardly as efficient as the tilting tank and also has the disadvantage that it requires power, but it has the advantage that the sand bin can be made 5 or 6 ft. higher.

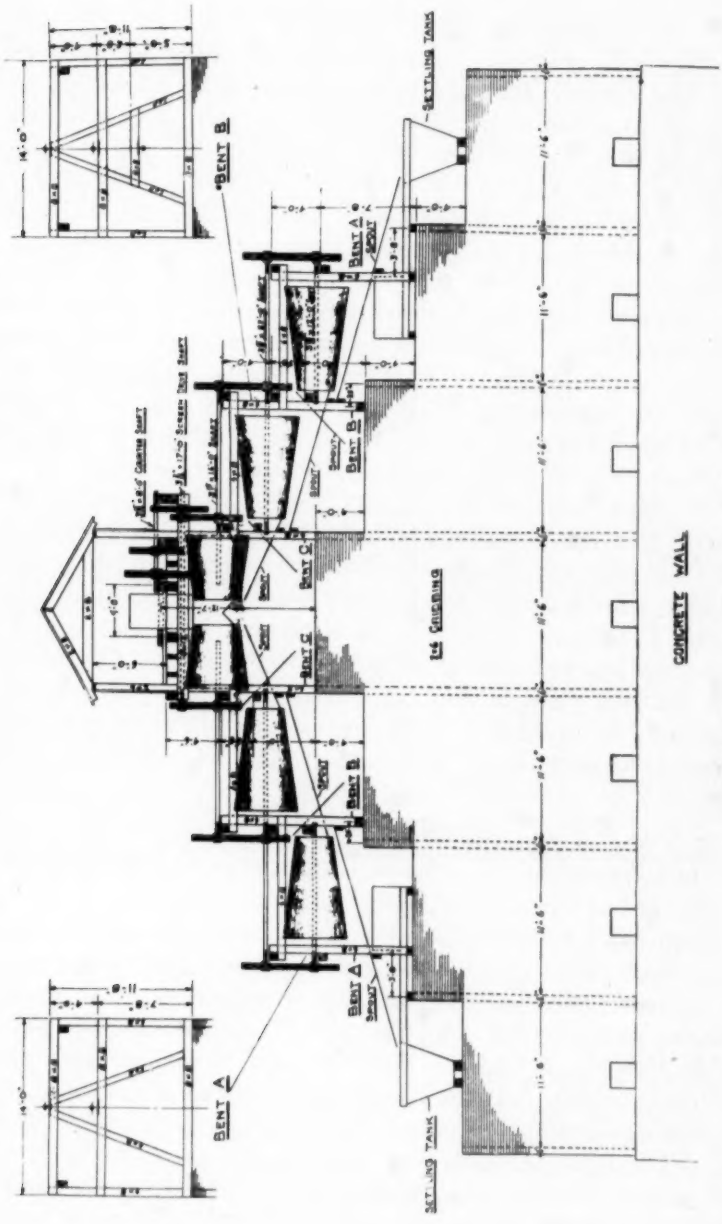

FIG. 7.—BINS OF CRIB CONSTRUCTION.

METHODS OF HANDLING.

The belt conveyor is almost universally used on the larger plants for carrying the material to the top of the bins, (Fig. 2). It is efficient, and if properly installed is very satisfactory for this purpose. It has been found, however, that the standard troughing idlers wear out very rapidly in handling this kind of material, and a pressed steel pulley was devised, with which to equip the idlers. Thus equipped they are able to carry large boulders, and being split, they are readily changed.

Belt conveyors are also used to convey material to the plant from the excavator. Drag-line excavators are also coming into use for plants of moderate capacity, Fig. 6. They have the advantage that one man at the hoisting engine can elevate and deliver the material to the plant. The plant shown is equipped with a machine which returns by gravity very rapidly.

If the material is below water, sand pumps are extensively used. This method gives the material a preliminary washing, but used with flat screens, which is the common practice, does not size accurately, and there is usually considerable sand in the gravel. The sand is not always clean and in some cases the material is pumped twice to accomplish the washing. The upkeep of a pumping plant is very high, as is also the power required. Some of the companies using pumps are installing revolving screens, with much better results than with flat screens.

BIN CONSTRUCTION.

In constructing bins, the skeleton construction is preferable to the cribbed construction. The skeleton design, Fig. 6, can be repaired much easier and requires less material to make. Solid plank walls, built up by spiking the planks flat-wise, one upon the other, is what is known as crib construction. The reason that this construction is not desirable is because so much water in the bins remains in the joints between the planks, and rots them very rapidly. If this construction is used, all joints should be well tarred (Fig. 7).

A few washing plants have reinforced concrete bins. Concrete makes excellent bins, and of course they are more permanent than bins made of wood. Circular steel tanks also make excel-

lent bins. The bin construction is usually a matter of preference with the purchaser.

LOADING DEVICES.

The sand and gravel is delivered into cars usually by steel counter-weighted spouts, although some bins are constructed over the track and the sand and gravel is dropped directly down into

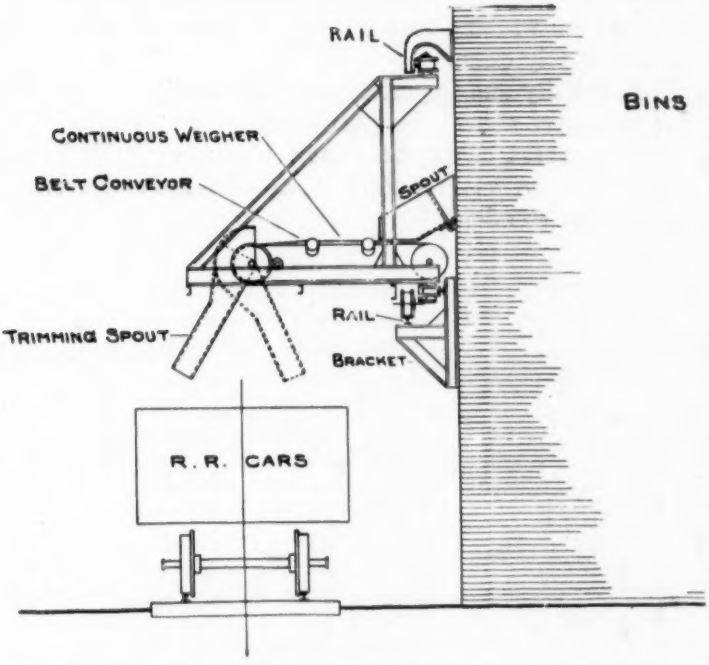

FIG. 8.—LOADING DEVICE WITH CONTINUOUS AUTOMATIC WEIGHER.

the cars. In shipping material in this manner there is always danger of overloading, and one is compelled to rely on railroad weights when shipping.

In order to weigh the material as it is loaded, there was lately designed a loading device, equipped with an automatic weigher. It consists of a short belt conveyor, mounted on a frame, very similar to a wall crane, Fig. 8, which travels on the face of the bins, and will stop in front of any gate opening. A